The Afghanistan Engineering Support Program assembled this deliverable. It is an approved, official USAID document. Budget information contained herein is for illustrative purposes. All policy, personal, financial, and procurement sensitive information has been removed. Additional information on the report can be obtained from Firouz Rooyani, Tetra Tech Sr. VP International Operations, (703) 387-2151.

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ENGINEERING SUPPORT PROGRAM

WORK ORDER WO-LT-0077 AMENDMENT 4
GARDEZ TO KHOST ROAD, BRIDGE No. 10
DESIGN ANALYSIS
FINAL DESIGN SUBMITTAL

November 12, 2014

This publication was produced for review by the United States Agency for International Development. It was prepared by Tetra Tech, Inc.

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TETRA TECH

November 12, 2014

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Re: Contract No. EDH-I-00-08-00027-00 / Task Order No. 1
Afghanistan Engineering Support Program (AESP)

WO-LT-0077 AMD 4 Gardez to Khost Road Bridge No. 10
Design Analysis Final Design Submittal

[redacted]

Tetra Tech is pleased to submit this Design Analysis Report, which accompanies the Bridge No. 10 Final Design submittal previously made under separate cover. Technical Specifications (to be used for both bridge #9 and Bridge #10) were previously submitted on October 28, 2014, and the final design drawings were also submitted on October 28, 2014.

Please contact us at your convenience should you have any questions or comments regarding these submittals.

Respectfully,
[redacted]

cc: Idrees Noori
Randal Leek

AFGHANISTAN ENGINEERING SUPPORT PROGRAM

Contract No. EDH-I-00-08-00027-00
Task Order No. 1
Work Order WO-LT-0077 Amendment 4

GARDEZ TO KHOST ROAD, BRIDGE No. 10
DESIGN ANALYSIS
FINAL DESIGN SUBMITTAL

November 12, 2014

Design Analysis

Introduction: The Design Analysis provides documentation of the basis for the design on the project. It is intended to describe the project requirements, identify governing codes and criteria being utilized, explain proposed design solutions and document other situations which affect the design. Engineering calculations are also included where appropriate.

The Design Analysis is organized by content and technical design discipline as follows:

Section 1 - Hydraulics

Section 2 - Geotechnical

Section 3 - Civil

Section 4 - Structural

Section 5 - Bill of Quantities

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Section 1
Hydraulics

Design Analysis

Discipline:	Hydraulics	Date:	October 12, 2014

Design Submittal: Final Design Submittal
Site Location: Bridge #10
Prepared By: Tetra Tech

I. General Summary:

Bridge #10 was located on the Gardez to Khost Road in Afghanistan, spanning over a tributary immediately west of a main river. The existing Bridge #10 was destroyed by floods and a temporary pipe culvert was installed. A new bridge crossing was designed in 2010 (by Others) to increase the hydraulic capacity of the crossing. Prior to construction of the new bridge, USAID requested that Tetra Tech perform a topographical survey, geotechnical investigation, geotechnical analysis, hydraulic modeling and structural analysis in order to determine if the 2010 Design is in conformance with the latest AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (AASHTO LRFD) standards and adequate based on the complete hydraulic, geotechnical and structural analyses. Tetra Tech performed this work and submitted the “Scour Analysis and Foundation Study” to USAID (dated July 15, 2014). The study recommended that the Bridge #10 crossing be redesigned. This “Final Design” submittal completes the redesign.

The proposed Bridge #10 is a two-span structure, similar in design and detailing to Bridge #09. The proposed bridge superstructure and substructure shall be constructed out of reinforced concrete. Approach roadway work is required to transition from the existing roadway to the bridge. The proposed bridge includes a concrete scour mattress for protection against scour.

II. Detailed Analysis

Hydraulic Analysis

At the Bridge #10 crossing, the Gardez to Khost Road crosses a tributary immediately west of a main river. Based on the topographic mapping, both the tributary and the main river are steep (average of 1% and 3%, respectively). Geotechnical data shows that the riverbed material is granular and non-plastic. Photographs in the survey report depict the river and tributary as braided, and cobble dominated systems with high width to depth ratios. It is possible these systems have high sediment supply, with the potential for excessive deposition both longitudinally and transversely. The banks appear to be erosive, likely a result of lateral movement of the river in response to significant flows. There are small settlements or individual homes located along the banks.

Tetra tech performed a hydraulic model for the proposed two-span bridge. The hydraulic capacity of the bridge and scour potential were evaluated using the estimated peak 50-year discharge on the tributary as stipulated by the project scope. The 2010 Design hydrologic analysis (prepared by Others) reports a 50-year discharge used for this analysis was 185.30 m^3/s. The watershed area for the tributary was reported as approximately 115.30 km^2.

Two hydraulic scenarios were assessed: 1) analysis of coincident peak flows on the main stem and on the tributary, producing the highest flow depths at the bridge, and 2) low flows in the main river and the 50-year discharge in the tributary producing the highest velocities calculated at the bridge. In addition, a hydraulic model of the main river was prepared to evaluate the scour potential at Bridge #10 due to the main river flow and other potential impacts on the bridge or the approaches. Using the hydrologic analysis in the 2010 Design report, the 50-year peak discharge of the main river was estimated. No peak discharge for the main river is reported at the location of Bridge #10, but peak discharge was reported for Bridges #9 and #11 which bracket the site.

To estimate the peak flow of the main river, the discharge and area for each crossing were plotted on a graph and fitted with a linear regression line passing through the origin. Data was used only if it was reported as calculated using HEC-HMS. An equation for the linear regression line was determined by the computer, using area as the variable. Using Soviet-era topographic data and the data within the report, the total drainage area of the main river at Bridge #10 was estimated to be approximately 529.13 km^2. The estimated peak flow of the main river at Bridge #10 is approximately 820 m^3/s.

A hydraulic analysis was conducted using HEC-RAS version 4.1.0, encoded using the topographic survey. The main river model consists of fourteen cross sections encoded at an interval between 10 and 60 meters. A Manning's *n* value of 0.045 was selected to represent the rocky, largely unvegetated condition within the channel and the overbank areas.

Along the tributary, eleven cross sections were encoded at an interval between 10 and 30 meters. A Manning's *n* value of 0.045 was selected to represent the rocky, unvegetated conditions in the river channel as shown in site photographs. A Manning's *n* value of 0.06 was used to represent some areas of vegetation and agriculture on each overbank area upstream of Bridge #10.

The proposed design is a two-span cast-in-place slab bridge with one pier. Each span is 16.8 meters long from centerline abutment bearing to centerline pier. Accounting for the width of the pier and the abutment, the total conveyance width 31.1 meters. The width of the bridge is approximately 11 meters. It was assumed for this analysis that the finished grade of the river bottom under the bridge will be approximately elevation 1816.31 meters upstream of the bridge and at approximately 1815.51 meters downstream of the bridge. The regraded elevations are located approximately 3 meters upstream and downstream of the bridge face, respectively. This is slightly lower than the existing channel grade, and is recommended to provide a continuous grade through the bridge.

In addition to grading in the vicinity of the bridge, a transition channel connecting the bridge opening and the existing tributary channel is recommended. The transition channel would tie in the bridge opening to the existing channel at a point approximately 50 meters upstream. The transition channel lowers the effective slope of the creek. The channel would have a variable bottom width with 3:1 H:V side slopes.

Hydraulic modeling results show velocities at the bridge approach of 2.19 m/s and shear stresses of approximately 74 N/m^2. Velocities through the bridge range from 2.98 m/s to 3.87 m/s. Velocities downstream of the bridge remain high, as the tributary meets the main river. The maximum water surface elevation at the upstream face of the bridge crossing is approximately at elevation 1818.54 meters. A summary of HEC-RAS results for the tributary is presented in Table 1. Results reported in the table are for peak flows on the tributary that are not coincident with a peak flow on the main river.

Table 1
Summary of HEC-RAS Calculations - Tributary

Cross Section	Water Surface El. (m)	Avg. Velocity (m/s)	Channel Shear Stress (N/m^2)
20*	1815.17	3.11	195.38
33*	1815.48	2.86	148.27
48*	1815.66	3.06	176.79
62*	1815.95	2.61	127.36
95	1816.94	3.51	225.53
	Bridge		
113	1818.54	2.19	74.24
134	1819.04	4.12	288.37
167	1821.30	4.26	275.04
194	1822.07	3.14	150.98
225	1822.23	4.19	290.87
254	1823.10	3.67	210.10

* Coincident with main river

A separate hydraulic model was prepared for the main river to evaluate the potential scour effects on Bridge #10 and potential overtopping of the approach roads or bridge. Modeling results for the main river showed that the approach roads and bridge have sufficient elevation above main river 50-year flood elevations. Regarding scour potential, evaluation of the model and topographic survey shows that the bridge location is outside the main flow areas of the river. This isolation from the main flow normally creates an ineffective flow area, which is characterized by very low flow velocities. The excavated channel flowline elevation is also higher than the main river, creating shallower flooding depths in the ineffective area. The scour potential at the bridge due to the main river is expected to be no greater than the scour potential due to the tributary flow.

The channel of the main river is expected to laterally migrate over time and is not predictable. The lateral migration can be mitigated along the road embankment through the use of riprap or other armoring systems. Riprap is recommended in the vicinity of Bridge 10 to protect the bridge and appurtenant structures from lateral migration of the main river.

A summary of the HEC-RAS results for the main river is presented in Table 2.

Table 2
Summary of HEC-RAS Calculations – Main River

Cross Section	Water Surface El. (m)	Avg. Velocity (m/s)
20*	1814.32	4.48
80*	1815.16	5.34
140*	1816.24	5.91
170*	1816.84	5.82
185*	1817.47	4.22
200*	1817.48	4.72
216	1817.59	4.61
243	1818.31	5.67
280	1818.90	5.76
310	1819.47	5.17
340	1820.28	3.67
400	1820.58	4.81
460	1822.21	5.14
520	1823.30	4.12

* Coincident with tributary

Channel Gradation

The subsurface investigation and testing conducted by Shawal GMTL is summarized in a report dated 29 May 2014. This report includes gradation logs at the two test pits performed in the channel. Based on subsequent conversations with Shawal GMTL, the "1.0 m" gradation logs are actually composite logs based on the samples they performed in depths from 0.0 to 3.0 meters. The gradation tests were based on a maximum sieve size of 75 mm (3 inches). Particles greater than 75 mm in diameter were weighed and accounted for in the reported gradations.

A summary of the d_{50} values for the test pits is presented in Table 3. The results of the gradation analyses show that minimum d_{50} for the test pit samples is approximately 5.7 mm and was used for the scour analysis. Values for d_{50} on the boring samples at all depths were not considered in this analysis. The method of obtaining the samples from depth makes it physically impossible to obtain particle sizes greater than 50 mm, which is not representative of the riverbed soil. Without the larger particle sizes in the sample, gradation results will be biased to the smaller particle sizes and will report a smaller d_{50} than normal. The smaller values were not considered to be representative of the overall stream system and the values were neglected for scour analysis. The selected d_{50} for the analysis was 5.7 mm. This value was selected because it was considered to be the smallest d_{50} for the soils that would normally aggrade or degrade during flood events.

Table 3
Summary of d_{50} (mm) for test pits

Test Pit ID	d_{50} (mm)
TP-1	5.7
TP-2	8.0

Bridge Scour Analysis

Scour potential at structures is a combination of long term scour, contraction scour, and localized scour at the abutments piers. Long term aggradation or degradation is the raising or lowering of the stream bed due to natural stream formation processes. Contraction scour can occur when flow is constricted from a wider floodplain into a narrower area, such as a bridge, and can occur over the entire streambed. Localized scour at abutments and piers is typically a result of vortices in flow. Localized scour is added to the contraction scour and long term scour. Contraction and localized scour analysis was performed using the HEC-RAS program.

Long term aggradation or degradation of the streambed may be considered in a scour analysis, but requires significant monitoring and analysis of the streambed over time in order to develop an estimate of long term aggradation and/or degradation. No data for this river was available for review, thus long term aggradation and/or degradation are not accounted for numerically in this analysis. Further, the potential for deposition or high sediment loading under high flow conditions is unknown and thus not considered in the overall hydraulic design-based recommendations. As previously noted, photographs in the survey report depict the river and tributary as braided, and cobble dominated systems with high width to depth ratios. It is possible these systems have high sediment supply, with the potential for excessive deposition both longitudinally and transversely. The banks appear to be erosive, likely a result of lateral movement of the river in response to significant flows. These observations lead to two recommendations: 1) provide bank stabilization in the vicinity of the bridge to stabilize the channel approaches, and 2) implement a monitoring program for changes in channel bed, including deposition, and perform maintenance to maintain the design dimension and elevations.

Contraction scour can either be clear water scour or live bed scour. Clear water scour can occur when the sediment in the uncontracted approach section is less than the sediment carrying capacity for that flow. Because this river is in a natural state, i.e. there are no dams or other factors to reduce sediment within the creek, and because it has high velocities, clear water scour was considered to be unlikely. Live bed scour, where some sediment load is carried into the crossing, was used for this analysis. This assumption is verified in HEC-RAS by the comparison of critical velocity, the velocity required to move the average size material, with the computed velocities. Calculations indicate the computed velocities exceed the critical values, thus supporting the live bed scour approach to this analysis.

Methods, equations, and coefficients for scour calculations are detailed in the HEC-RAS *Hydraulic Reference Manual* and HEC-18 *Estimating Scour at Bridges*. HEC-RAS utilizes Laursen's live-bed contraction scour analysis. Pier scour and abutment scour can be calculated using one of several methods available in HEC-RAS. The Colorado State University (CSU) equation was selected for estimating pier scour and the Froehlich Equation was selected for estimating abutment scour. No wood debris accumulation was considered in the pier width based on the lack of timber observed in the photos.

A summary of the calculated scour results is presented in Table 4. The values for the top of footing of the abutments summarized in the table below was calculated as the minimum channel elevation, located at the downstream end of the bridge, minus the scour depth. The maximum top of footing elevation for the pier was estimated by the model and differs from the modeled result included in the appendices. The scour depth from the model is estimated using the equations in HEC-18. However, the scour cavities from the abutments are larger than the modeled pier scour depth. The modeled abutment scour cavities have sufficient depth that the cavity is larger than the pier scour cavity. In addition, the material remaining under the pier is expected to be insufficient for structural support. It is recommended to establish the top of pier elevation as the same elevation for the abutments.

Table 4
Proposed Design Scour Depths

	West Abutment (left)	**Pier**	**East Abutment (right)**
Total scour depth	9.48 m	1.09 m	9.48 m
Minimum channel elevation (downstream side of bridge)	1815.51		
Maximum top of footing elevation for scour protection	1806.03 m	1806.03 m	1806.03 m

Generally, if the flow velocity in the stream is less than the threshold flow velocity for mobilization of bed material, a riprap blanket around the pier might help reduce scour. However, in the case of Bridge #10, the channel velocities are greater than that required for mobilization so the use of riprap at the piers is discouraged because the loose riprap will break up (dissipate) due to the secondary flow patterns at and around the piers, and sink down into the streambed offering no protection from scour at the piers.

Scour Protection Design

Several alternatives were considered for protection of the piers and abutments from the calculated scour depths. Alternatives that were evaluated include deeper spread footing foundations, drilled foundations, concrete armoring of the channel, and armoring the channel with articulated concrete blocks. Evaluations included constructability, cost, availability of skilled labor and equipment and schedule. Similar to our experience with Bridge #09, **a concrete apron is recommended to armor the channel.** The concrete apron should include downward sloping key walls to protect the apron from undermining.

The concrete apron is intended to prevent the formation of scour holes at the pier and abutment. By covering the riverbed soil, scour holes are not able to propagate out from the structure where they form. Some local scour is anticipated at the edges of the apron where flow transitions back to normal river flows. No research has been done for this specific type of application. An estimate for this local scour was adapted from existing methods to determine the approximate depth.

A calculation for general scour using *Technical Supplement 14B* of the National Engineering Handbook was used to estimate general scour depth. The general river scour estimate is noted as equation TS14B-23 in the publication. The equation for general scour is:

$$z_t = K Q_d^a W_f^b d_{50}^c$$

Where:

z_t	maximum scour depth (m)
K	coefficient from table TS14B-8
Q_d	design discharge (m^3/s)
W_f	flow width (m)
d_{50}	median size of bed material (mm)
a,b,c	exponents from table TS14B-8

Coefficients and exponents in the equation are determined by the general geometry of the river. In this location, the “right angle” coefficients and exponents were selected because the river does turn approximately 90 degrees just downstream of the bridge. Coefficients also vary based on experimental data by two researchers (Lacey and Blench). For the purposes of this evaluation, both data sets are utilized for calculations. The d_{50} of the material used for this calculation was approximately 5.7 mm, which is the average d_{50} determined from laboratory data.

Using the selected parameters above and data from the HEC-RAS model, the estimated scour depth using the Lacey relations was approximately 1.7 meters. The estimated scour depth using the Blench relations is approximately 3.0 meters.

The calculated scour depths show satisfactory correspondence between the two methods. To provide a factor of safety, the sloped key walls for the apron are recommended to be set to a depth of 3.0 meters below the edge of apron.

Tetra Tech evaluated the potential for uplift of the concrete mat at varying flow conditions across the mat. Velocities for each flow condition were used to determine the uplift force that the mat would experience. Forces that were calculated to counteract the uplift forces were the weight of the mat itself and the weight of the water above the concrete mat. The typical factor of safety used for uplift resistance is 1.5.

The nominal mat thickness used in the analysis was 0.20 meters (8 inches). Calculations for uplift for the apron were based on the assumption that the channel would be graded as described in preceding sections of this report. Results of the uplift calculations show that this apron thickness should be sufficient to resist uplift forces. See attached design calculations.

III. References

- AASHTO "LRFD Bridge Design Specifications." 6th Edition, 2012
- "Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Scour Analysis and Foundation Study" dated July 15, 2014 (prepared by Tetra Tech)
- "Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Geotechnical Report" dated June 17, 2014 (prepared by Tetra Tech)
- U.S. Department of Transportation, *Hydraulic Engineering Circular No. 18 - Evaluating Scour at Bridges*, April 2012.
- USBR Report DSO-07-07. *Uplift and Crack Resistance Resulting from High Velocity Discharges Over Open Offset Joints*. Figure 11. December 2007.
- US Department of Agriculture, Natural Resource Conservation Services. *National Engineering Handbook, Part 654, Technical Bulletin 14B – Scour Calculations*. August 2007.

IV. List of Attachments:

Calculations

- Scour Analysis and Gradation Analysis
- HEC-RAS Results
- Scour Analysis from HECRAS
- Riprap Stability Calculations
- General Scour Calculations – With Concrete Apron
- Uplift Resistance Calculations

Calculations

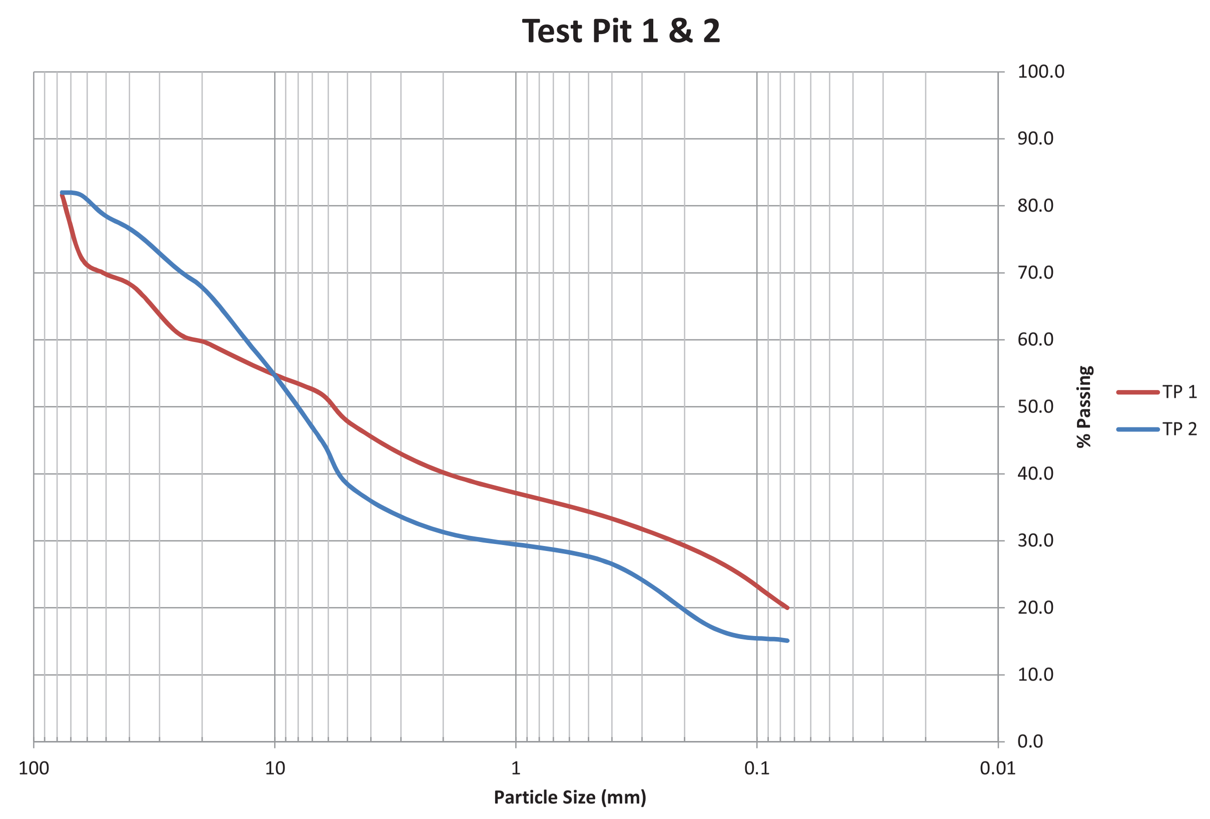
Gardez-Khost Road Bridge #10

SAMPLE TP 01 and 02 - Composite Sample (all depths)

Sieve Name	Sieve Size (mm)	TP - 1	TP -2
		Percent Passing at Specified Depth	
3"	76.2	81.6	82.0
2.5"	63.5	72.3	81.6
2"	50.8	69.9	78.6
1.5"	38.1	67.8	76.1
1"	25.4	61.1	70.6
3/4"	19.05	59.5	67.1
1/2"	12.7	56.4	59.3
3/8"	9.53	54.5	53.7
1/4"	6.35	51.8	44.8
No. 4	4.75	47.3	37.8
No. 10	2	40.2	31.3
No. 40	0.425	33.6	26.9
No. 100	0.150	27.2	16.9
No. 200	0.075	20.0	15.1

D50 for each depth (mm)

Test Pit	D50 (mm)
TP 1	5.7
TP 2	8



Geotechnical data summarized from Geotechnical Report Addendum by Shawal Geotechnical Laboratory

Gardez-Khost Road Bridge #10

HEC-RAS Results

Proposed Tributary Hydraulic Model

Model Features:

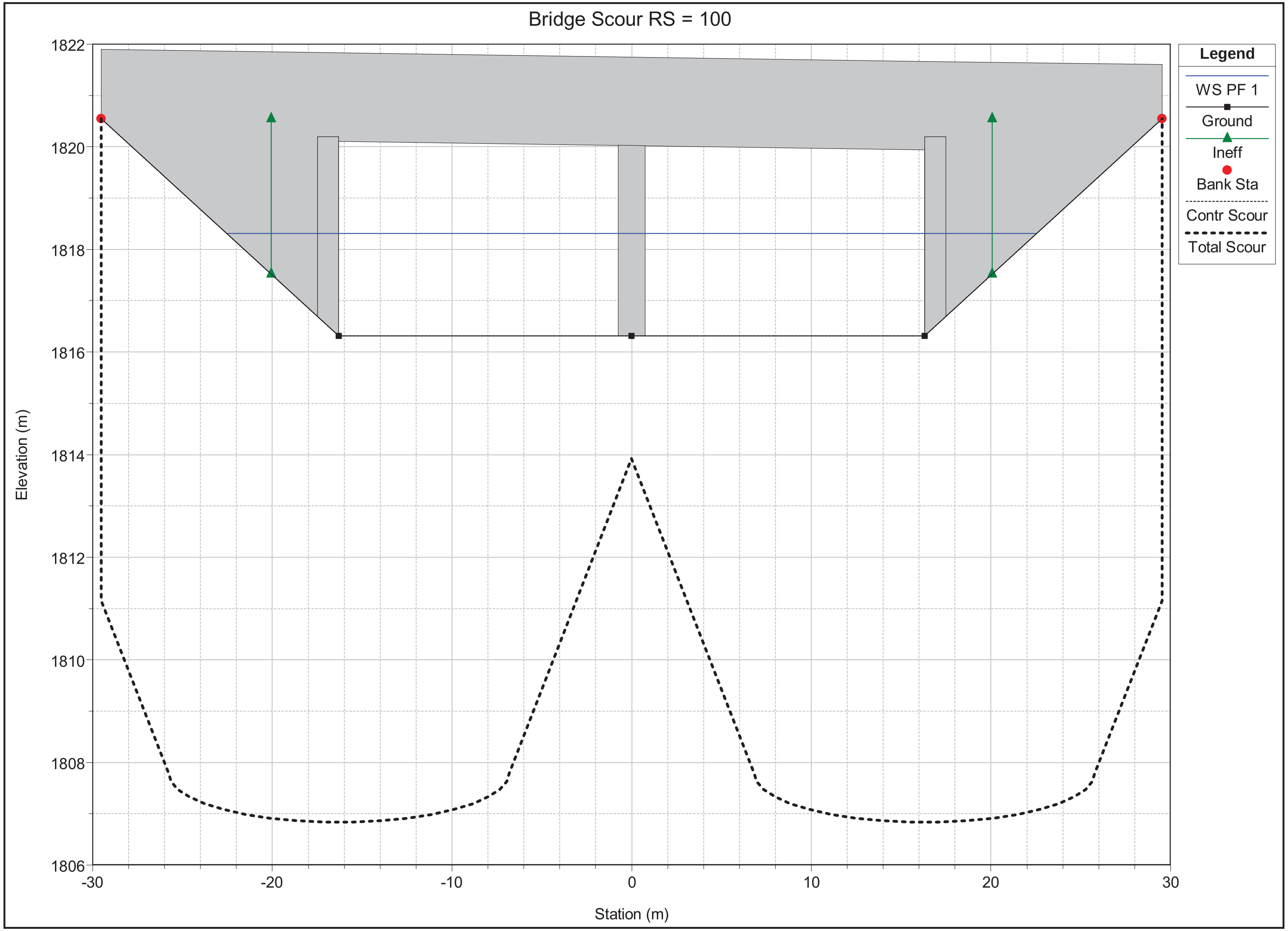
2-span bridge per Tetra Tech 2014 Design

Channel graded to bridge approach

Profile 1 - Assumes no flooding in Main River

Profile 2 - Assumes coinicident peak flooding in Main River

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Tributary	20	PF 1	185.3	1813.26	1815.17	1815.17	1815.66	0.020866	3.11	59.53	62	1.01
Tributary	20	PF 2	185.3	1813.26	1816.84	1815.17	1816.89	0.000708	1.02	180.86	78.03	0.21
Tributary	33	PF 1	185.3	1813.77	1815.48		1815.86	0.011431	2.86	71.19	67.76	0.79
Tributary	33	PF 2	185.3	1813.77	1817.47		1817.5	0.000343	0.91	232.67	86.68	0.16
Tributary	48	PF 1	185.3	1813.99	1815.66	1815.43	1816.07	0.015392	3.06	67.42	70.29	0.9
Tributary	48	PF 2	185.3	1813.99	1817.48		1817.52	0.000469	0.92	210.59	84.64	0.18
Tributary	62	PF 1	185.3	1814.41	1815.95		1816.26	0.010758	2.61	75.69	71.32	0.76
Tributary	62	PF 2	185.3	1814.41	1817.59		1817.63	0.000505	1	202.37	81.95	0.19
Tributary	95	PF 1	185.3	1815.51	1816.94	1816.94	1817.56	0.018119	3.51	52.84	41.49	0.99
Tributary	95	PF 2	185.3	1815.51	1817.44		1817.76	0.006057	2.52	73.41	44.61	0.6
Tributary	100		Bridge									
Tributary	113	PF 1	185.3	1816.31	1818.54	1817.72	1818.78	0.003618	2.19	84.74	46.48	0.48
Tributary	113	PF 2	185.3	1816.31	1818.42	1817.72	1818.69	0.004376	2.32	80.04	45.74	0.52
Tributary	134	PF 1	185.3	1817.24	1819.04	1819.04	1819.91	0.018421	4.12	44.98	25.58	0.99
Tributary	134	PF 2	185.3	1817.24	1819.04	1819.04	1819.91	0.018421	4.12	44.98	25.58	0.99
Tributary	167	PF 1	185.3	1818.74	1821.3	1821.3	1822.01	0.01254	4.26	59.07	95.97	0.89
Tributary	167	PF 2	185.3	1818.74	1821.3	1821.3	1822.01	0.01254	4.26	59.07	95.97	0.89
Tributary	194	PF 1	185.3	1819.5	1822.07		1822.31	0.007017	3.14	104.46	88.34	0.67
Tributary	194	PF 2	185.3	1819.5	1822.07		1822.31	0.007017	3.14	104.46	88.34	0.67
Tributary	225	PF 1	185.3	1820.26	1822.23	1822.23	1822.72	0.017274	4.19	71.93	70.21	1.01
Tributary	225	PF 2	185.3	1820.26	1822.23	1822.23	1822.72	0.017274	4.19	71.93	70.21	1.01
Tributary	254	PF 1	185.3	1820.59	1823.1	1823.1	1823.52	0.010362	3.67	83.87	85.75	0.81
Tributary	254	PF 2	185.3	1820.59	1823.1	1823.1	1823.52	0.010362	3.67	83.87	85.75	0.81


Bridge Scour RS = 100
Legend
WS PF 1
Ground
Ineff
Bank Sta
Contr Scour
Total Scour
Elevation (m)
1822
1820
1818
1816
1814
1812
1810
1808
1806
-30
-20
-10
0
10
20
30
Station (m)

Contraction Scour

		Left	Channel	Right
Input Data				
	Average Depth (m):		1.76	
	Approach Velocity (m/s):		4.12	
	Br Average Depth (m):		2.00	
	BR Opening Flow (m3/s):		185.30	
	BR Top WD (m):		31.10	
	Grain Size D50 (mm):		5.70	
	Approach Flow (m3/s):		185.30	
	Approach Top WD (m):		25.58	
	K1 Coefficient:		0.640	
Results				
	Scour Depth Ys (m):		0.00	
	Critical Velocity (m/s):		1.21	
	Equation:		Live	

Pier Scour

All piers have the same scour depth

Input Data		
	Pier Shape:	Round nose
	Pier Width (m):	1.50
	Grain Size D50 (mm):	5.70000
	Depth Upstream (m):	2.11
	Velocity Upstream (m/s):	2.19
	K1 Nose Shape:	1.00
	Pier Angle:	0.00
	Pier Length (m):	10.95
	K2 Angle Coef:	1.00
	K3 Bed Cond Coef:	1.10
	Grain Size D90 (mm):	100.00000
	K4 Armouring Coef:	0.40
Results		
	Scour Depth Ys (m):	1.09
	Froude #:	0.48
	Equation:	CSU equation

Abutment Scour

		Left	Right
Input Data			
	Station at Toe (m):	-16.30	16.30
	Toe Sta at appr (m):	94.09	93.52
	Abutment Length (m):	13.07	13.07
	Depth at Toe (m):	2.22	2.22
	K1 Shape Coef:	0.82 - Vert. with wing walls	
	Degree of Skew (degrees):	90.00	90.00
	K2 Skew Coef:	1.00	1.00
	Projected Length L' (m):	13.07	13.07
	Avg Depth Obstructed Ya (m):	1.76	1.76
	Flow Obstructed Qe (m3/s):	94.69	94.69
	Area Obstructed Ae (m2):	22.99	22.99
Results			
	Scour Depth Ys (m):	9.48	9.48
	Qe/Ae = Ve:	4.12	4.12
	Froude #:	0.99	0.99

General Scour Calculations						
Bridge 10 Concrete Apron Scour Calculations						
National Engineering Handbook, Part 654, Technical Supplement 14B						
Equation TS14B-23						
	Qd (m3/s)	185.3				
	Wf (m)	31.06				
	d50 (mm)	5.7				
Lacey	K	0.389		Right Angle Bend		
	a	0.333333				
	b	0				
	c	-0.16667				
Blench	K	1.105		Right Angle Bend		
	a	0.666667				
	b	-0.66667				
	c	-0.1092				
General Scour						
Lacey	Z (m)	1.659				
Blench	Z (m)	3.006				

	Bridge 10 Uplift Resistance Calculations										
	Comparison of Uplift Pressure v. Weight of Water+Concrete										
				Unit Weight Water	**9.81**	**kN/m3**					
				Unit Weight Concrete	**23.6**	**kN/m3**					
				Area	**1**	**m2**					
				Concrete Thickness	**8**	**in**					
				Concrete Thickness	**0.2032**	**m**					
				Weight of Concrete	**4.80**	**kN/m2**					
Minimum Desired Factor of Safety for Design Flow and Lower Flows					**1.5**						
	BRIDGE 100 UPSTREAM										
	Q (m3/2)	**WSE**	**Mat Elev.**	**Depth of Water (m)**	**Weight of Water (kN/m2)**	**Velocity (m/s)**	**Velocity (ft/s)**	**Uplift Head (ft)**	**Uplift Head (m)**	**Uplift Pressure (kN/m2)**	**Factor of Safety**
50-Yr	185.3	1818.31	1816.31	2	19.6	2.98	9.8	2.2	0.7	6.4	3.8
	150	1818.46	1816.31	2.15	21.1	2.73	9.0	1.9	0.6	5.6	4.6
	100	1817.71	1816.31	1.4	13.7	2.3	7.5	1.4	0.4	4.3	4.3
	75	1817.5	1816.31	1.19	11.7	2.03	6.7	1.2	0.4	3.5	4.7
	50	1816.95	1816.31	0.64	6.3	2.5	8.2	1.6	0.5	4.9	2.3
	BRIDGE 100 DOWNSTREAM										
	Q (m3/2)	**WSE**	**Mat Elev.**	**Depth of Water (m)**	**Weight of Water (kN/m2)**	**Velocity (m/s)**	**Velocity (ft/s)**	**Uplift Head (ft)**	**Uplift Head (m)**	**Uplift Pressure (kN/m2)**	**Factor of Safety**
	185.3	1817.81	1815.51	2.3	22.6	3.87	12.7	3.3	1.0	9.9	2.8
	150	1816.85	1815.51	1.34	13.1	3.61	11.8	2.9	0.9	8.8	2.0
	100	1816.53	1815.51	1.02	10.0	3.16	10.4	2.4	0.7	7.1	2.1
	75	1816.35	1815.51	0.84	8.2	2.87	9.4	2.0	0.6	6.1	2.2
	50	1816.19	1815.51	0.68	6.7	2.35	7.7	1.5	0.5	4.4	2.6
	SECTION 95										
	Q (m3/2)	**WSE**	**Mat Elev.**	**Depth of Water (m)**	**Weight of Water (kN/m2)**	**Velocity (m/s)**	**Velocity (ft/s)**	**Uplift Head (ft)**	**Uplift Head (m)**	**Uplift Pressure (kN/m2)**	**Factor of Safety**
	185.3	1816.94	1815.5	1.44	14.1	3.51	11.5	2.8	0.9	8.4	2.3
	150	1816.76	1815.5	1.26	12.4	3.29	10.8	2.5	0.8	7.5	2.3
	100	1816.47	1815.5	0.97	9.5	2.94	9.6	2.1	0.6	6.3	2.3
	75	1816.31	1815.5	0.81	7.9	2.68	8.8	1.8	0.6	5.4	2.3
	50	1816.12	1815.5	0.62	6.1	2.37	7.8	1.5	0.5	4.5	2.4
	SECTION 113										
	Q (m3/2)	**WSE**	**Mat Elev.**	**Depth of Water (m)**	**Weight of Water (kN/m2)**	**Velocity (m/s)**	**Velocity (ft/s)**	**Uplift Head (ft)**	**Uplift Head (m)**	**Uplift Pressure (kN/m2)**	**Factor of Safety**
	185.3	1818.54	1816.3	2.24	22.0	2.19	7.2	1.3	0.4	4.0	6.8
	150	1818.27	1816.3	1.97	19.3	2.03	6.7	1.2	0.4	3.5	6.8
	100	1817.84	1816.3	1.54	15.1	1.76	5.8	1.0	0.3	2.8	7.0
	75	1817.6	1816.3	1.3	12.8	1.59	5.2	0.8	0.2	2.4	7.2
	50	1817.21	1816.3	0.91	8.9	1.57	5.2	0.8	0.2	2.4	5.7

Section 2
Geotechnical

Design Analysis

Discipline:	Geotechnical	Date:	October 12, 2014

Design Submittal: Final Design Submittal
Site Location: Bridge #10
Prepared By: Tetra Tech

I. General Summary:

Bridge #10 was located on the Gardez to Khost Road in Afghanistan, spanning over a tributary immediately west of a main river. The existing Bridge #10 was destroyed by floods and a temporary pipe culvert was installed. A new bridge crossing was designed in 2010 (by Others) to increase the hydraulic capacity of the crossing. Prior to construction of the new bridge, USAID requested that Tetra Tech perform a topographical survey, geotechnical investigation, geotechnical analysis, hydraulic modeling and structural analysis in order to determine if the 2010 Design is in conformance with the latest AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (AASHTO LRFD) standards and adequate based on the complete hydraulic, geotechnical and structural analyses. Tetra Tech performed this work and submitted the “Scour Analysis and Foundation Study” to USAID (dated July 15, 2014). The study recommended that the Bridge #10 crossing be redesigned. This “Final Design” submittal completes the redesign.

The proposed Bridge #10 is a two-span structure, similar in design and detailing to Bridge #09. The proposed bridge superstructure and substructure shall be constructed out of reinforced concrete. Approach roadway work is required to transition from the existing roadway to the bridge. The proposed bridge includes a concrete scour mattress for protection against scour.

II. Detailed Analysis:

Geotechnical Investigation

The Geotechnical investigation was performed by Shawal Geotechnical Engineering/ Materials Testing Laboratory (Shawal GMTL). The Geotechnical investigation included borings, test pits, sampling, field testing and laboratory testing. A summary of the field investigation and the results of the testing are provided in a report entitled “Soil Test Results Reports for Gardez to Khost Bridge #10, Khost Province, Afghanistan” dated 29 May 2014.

As noted in their report, their investigation included three boreholes with a completion depth of 15 meters below the existing ground surface. One borehole was drilled at each of the bridge supports (abutment & pier footings) and two test pits were excavated in the channel. Laboratory analyses of the samples were also performed to evaluate engineering characteristics of the bridge’s subgrade.

Soil samples were obtained during the drilling operations by driving a Standard Split Spoon sampler at 1 meter intervals. The sampler was driven with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler were recorded in accordance with the Standard Penetration Test (SPT) per ASTM D1586. The SPT values are useful in evaluating the relative density and consistency of the soils. The SPT values indicated the alluvial soils generally range from medium dense to very dense. In some cases, refusal was listed at areas logged as boulders. The soil samples recovered during the drilling operations were tested for in-situ moisture content, Atterberg Limits, and gradation. In addition, one soil sample from each boring was subjected to direct shear strength testing per ASTM standard D3080.

Larger bulk soil samples were obtained from the test pits excavated in the channel. These samples were tested for in-situ moisture, modified Proctor moisture / density relationships, California Bearing Ratio on samples compacted to 95% of modified Proctor density, and gradation analyses. In addition, in-situ moisture and density were measured at each test pit using sand cone methods. Gradation testing on the test pit samples is considered more representative due to the coarseness of the alluvium.

The Shawal GMTL report reflects that the subsurface material is non-plastic to low plastic and medium dense to very dense, generally coarse alluvium. The alluvial clasts range in size from sand to cobble and boulder sized material and are locally silty and/or clayey. Groundwater was encountered approximately 4.0 m below the channel bed.

Review of Geotechnical Data

Although the geotechnical report prepared by Shawal GMTL contained geotechnical design parameters and recommendations, Tetra Tech independently performed calculations to determine the design parameters and recommendations in accordance with AASHTO LRFD since the subsequent bridge evaluation (see Section 6.0) was performed in accordance with AASHTO LRFD.

Tetra Tech's full recommendations, including ultimate bearing resistance calculations and a settlement analysis, can be found in "Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Geotechnical Report" dated June 17, 2014. These calculations were based on soil property values that are typical of those soils encountered in the soil boring logs, the gradation analysis of the test pits performed in the channel and the following assumptions:

- Used AASHTO LRFD methodology considering the shape of the foundation, depth of embedment, and the shearing resistance of the soil above the foundation.
- Assumed bearing soil is fully saturated
- Assumed cohesion value is zero since the soils encountered underlying the bridge foundation are granular and non-plastic in nature.
- Used footing geometry as defined in the structural plans.

Recommendations

Tetra Tech performed geotechnical analyses based on the three borings and two test pits performed during the geotechnical investigation, and the applied loads from the 2010 Design, as calculated by Tetra Tech. The geotechnical calculations, performed in accordance with AASHTO LRFD resulted in calculated settlements less than 10 mm.

Tetra Tech recommends that Bridge #10 be supported on shallow foundations (spread footings) at the abutments and piers. The bottom of footings shall be located a minimum of 1.0 m below grade due to frost concerns.

III. Basis of Design

The abutments, pier and retaining walls should be designed in accordance with the following design parameters:

- Groundwater level at channel grade
- Weight of Soil = 20.5 kN/m^3 (130.4 pcf)
- Angle of Internal Friction = 33 degrees
- Ko = 0.46
- Ka = 0.29
- Kp = 3.39
- Coefficient of Friction for Sliding = 0.57
- Bearing Resistance for the Abutments:
  - Nominal Resistance: 1874 kN/m^2 (39.1 ksf)
  - Factored Bearing Resistance:
    - Non-Seismic Load Cases (ø=0.45) 843 kN/m^2 (17.6 ksf)
    - Seismic Load Cases (ø=1.0) 1874 kN/m^2 (39.1 ksf)

- Bearing Resistance for the Retaining Walls:
  - Nominal Resistance: 1050 kN/m^2 (21.9 ksf)
  - Factored Bearing Resistance:
    - Non-Seismic Load Cases (ø=0.45) 472 kN/m^2 (9.9 ksf)
    - Seismic Load Cases (ø=1.0) 1050 kN/m^2 (21.9 ksf)
- Bearing Resistance for the Pier:
  - Nominal Resistance: 788 kN/m^2 (16.5 ksf)
  - Factored Bearing Resistance:
    - Non-Seismic Load Cases (ø=0.45) 355 kN/m^2 (7.4 ksf)
    - Seismic Load Cases (ø=1.0) 788 kN/m^2 (16.5 ksf)

IV. Material Properties

See Section III, Basis of Design, for Tetra Tech’s recommended soil properties based on the geotechnical investigation.

V. References

- AASHTO “LRFD Bridge Design Specifications” 6th Edition, 2012
- “Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Scour Analysis and Foundation Study” dated July 15, 2014 (prepared by Tetra Tech)
- “Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Geotechnical Report” dated June 17, 2014 (prepared by Tetra Tech)
- Das, Braja M. “Principles of Foundation Engineering.” Sixth Edition, 2007
- Das, Braja M. “Fundamentals of Geotechnical Engineering.” Second Edition, 2005.
- Holtz, Kovacs, and Sheahan. “An Introduction to Geotechnical Engineering.” Second Edition, 2010.

VI. List of Attachments:

Calculations

Calculations

Bearing Resistance Abutments

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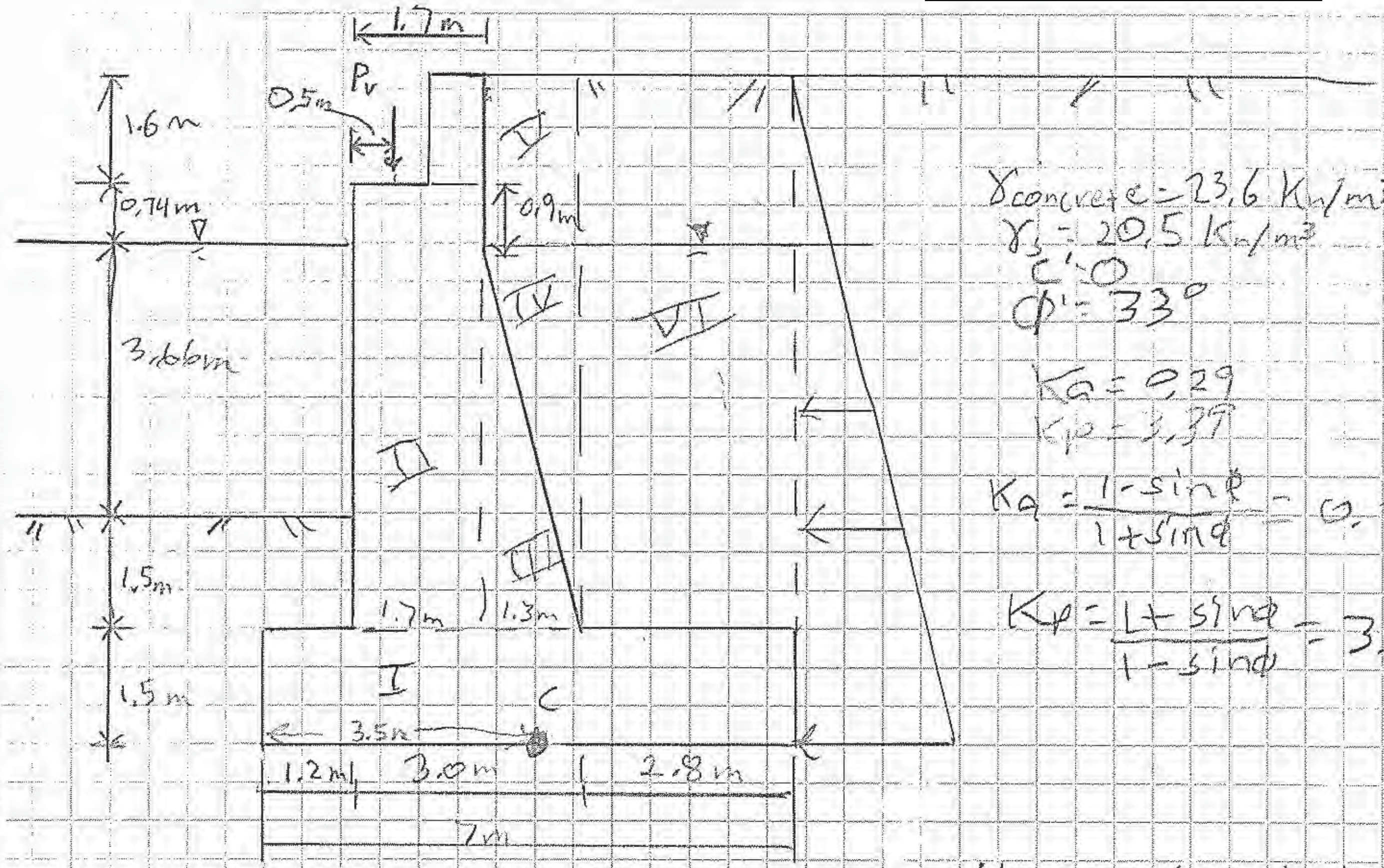
Client: USAID — Job No.: — Sheet 1 of 5

Description: Bearing Capacity Abutment Bridge 10 — Designed By: GL — Date: 10-2-14

Checked By: Ron Versaw — Date: 10/4/14



$\gamma_{concrete} = 23.6$ Kn/m³

$\gamma_s = 20.5$ Kn/m³

$c' = 0$

$\phi' = 33°$

$K_a = 0.29$

$K_p = 3.39$

$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.29$

$K_p = \frac{1+\sin\phi}{1-\sin\phi} = 3.39$

Abutment length = 13.45 m

Bearing length = 11.25m

$P_v = 19.02$ K/LF = 277.6 Kn/m

Region	Area (m²)	γ (Kn/m³)	W (Kn/m)	Mom Arm (m)	M (KN m/m) About pt. C
I	10.5	23.6	247.8	0	0
II	10.03	23.6	236.7	1.45	343.2
III	3.35	23.6	79.2	0.17	13.5
IV	3.35	20.5	68.7	-0.27	-18.5
V	3.04	20.5	62.4	-0.05	-3.1
VI	21.0	20.5	430.5	-2.1	-904.1
P_v	—	—	277.6	1.8	499.7
F_{A1}	—	—	16.3	5.2	84.8
F_{A2}	—	—	85.1	2.22	188.1
F_{A3}	—	—	217.6	2.22	483.1
F_{A4}	—	—	92.6	3.33	308.4

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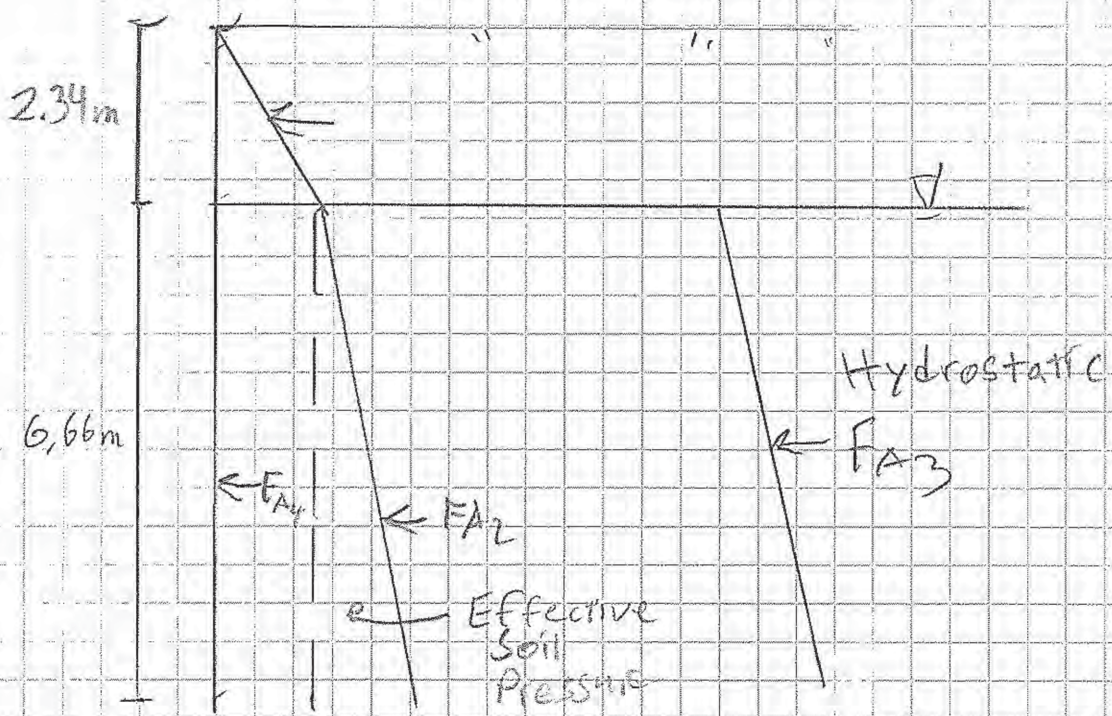
Client: USAID Job No.: Sheet 2 of 5

Description: Bearing Capacity Abutment Bridge 10 Designed By: GL Date: 10-3-14

Checked By: Date:

Calculation of Active Earth and Hydrostatic Pressures



$F_A = 0.5(\gamma)(H^2)(K_a)$

$F_{A_1} = 0.5(20.5\,kN/m^3)(2.34^2\,m)(0.29) = 16.3\,kN/m$

moment arm $= 6.66 + \frac{1}{3}(2.34\,m) = 5.2\,m$

$F_{A_2} = F_{A_1} + 0.5(20.5\,kN/m^3 - 9.81\,kN/m^3)(6.66^2\,m)(0.29) = 85.1\,kN/m$

moment arm $F_{A_2} = \frac{1}{3}(6.66\,m) = 2.22$

$F_{A_3} = 0.5(9.81\,kN/m^3)(6.66\,m)^2 = 217.6\,kN/m$

Moment arm $F_{A_3} = F_{A_2} = 2.22$

$F_{A_4} = (2.34\,m)(20.5)(6.66\,m)(0.29) = 92.6\,kN/m$

moment arm $= \frac{1}{2}(6.66\,m) = 3.33$

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Client: USAID Job No.: Sheet 3 of 5

Description: Bearing capacity Designed By: GL Date: 10-3-11

Abutment Bridge 10 Checked By: Date:

Eccentricity

$$e = \frac{343.2 + 13.5 - 18.5 - 3.1 - 904.1 + 499.7 + 84.8 + 185.1 + 453.1 + 3 \ldots}{2479 + 236.7 + 79.2 + 62.4 + 62.4 + 430.5 + 272.6}$$

$$e = \frac{995.1 \ \mathrm{Kn\,m/m}}{1{,}402.9 \ \mathrm{Kn/m}} = 0.71\,\mathrm{m}$$

vertical stress

$$\sigma_v = \frac{\Sigma v}{B - 2e} = \frac{1{,}402.9}{7 - 2(.71)} = 251.4 \ \frac{\mathrm{Kn}}{\mathrm{m}^2}$$

Soil Bearing capacity

Assumptions

- cohesionless soil
- effective stress analysis
- drained strength parameters
- groundwater at surface
- bottom of footing 3 m below scour
- eccentric load = eccentric footing dimension
- service I load conditions

q_n : nominal bearing resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 B N_{\gamma m} C_{w\gamma}$$

Since cohesion = 0 c = 0

$$q_n = \gamma D_f N_{qm} C_{wq} + 0.5 B N_{\gamma m} C_{w\gamma}$$

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Client: USAID — Job No.: — Sheet 4 of 5

Description: Bearing capacity Abutment Bridge 10 — Designed By: GL — Date: 10-3-14

Checked By: — Date:

$N_{qm} = N_q s_q d_q i_q$

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$

Where N_q = bearing capacity for surcharge (table 10.6.3.1.2a-1)

$\phi' = 33°$ $N_q = 26.1$

N_γ bearing capacity soil unit weight (table 10.6.3.1.2a-1)

$N_\gamma = 35.2$

γ total moist soil unit weight = 20.5 kN/m³

D_f = footing embedment depth = 3.0 m

Eccentricity already calculated = 0.71 m

$B' = B - 2e$

$B' = 7 - 2(0.71m) = 5.58m$

$L' = L = 11.25m$

$A' = L'B' = 11.25_m(5.58_m) = 62.78m^2$

$C_{wq} = C_{w\gamma}$ = correction factor for groundwater depths

table 10.6.3.1.2a-2

$C_{wq} = C_{w\gamma} = 0.5$

s_q = footing shape factor table 10.6.3.1.2a-3

$s_q = 1 + \left(\frac{B}{L}\tan\phi\right) = 1 + \frac{5.58}{11.25}\tan 33° = 1.32$

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Client: USAID Job No.: Sheet 5 of 5

Description: Bearing capacity Abutment of Bridge 10 Designed By: GL Date: 10-2-14

Checked By: Date:

S_γ = footing shape factor, table 10.6.3.1.2a-3

$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{5.58}{11.25}\right) = 0.81$$

load inclination factors = 1 = i_q and i_γ

d_q = correction factor to account for soil around fo[...]

$d_q = 1$ conservative

⇒ Find N_{qm} and $N_{\gamma m}$

$$N_{qm} = N_q S_q d_q i_q = (26.1)(1.32)(1)(1) = 34.45$$

$$N_{\gamma m} = N_\gamma S_\gamma i_\gamma = (35.2)(0.81)(1) = 28.5$$

Find q_m

$$q_m = \gamma D_q N_{qm} C_{wq} + 0.5\,\gamma B N_{\gamma m} C_{w\gamma}$$

$$q_m = (20.5\,\tfrac{Kn}{m^3})(3m)(34.45)(0.5) + (0.5)(20.5\,\tfrac{Kn}{m^3})(5.58m)(28.5)(0.5)$$

$$q_m = 1{,}059\,\tfrac{Kn}{m^2} + 815\,\tfrac{Kn}{m^2} = 1{,}874\,\tfrac{KN}{m^2}$$

$$q_r = \phi q_n \quad \phi = 0.45 \quad 0.45(1{,}874\,\tfrac{KN}{m^2}) = 843\,\tfrac{Kn}{m^2}$$

$q_r > \sigma_v$ ⇒ OK

Bearing Resistance
Retaining Walls

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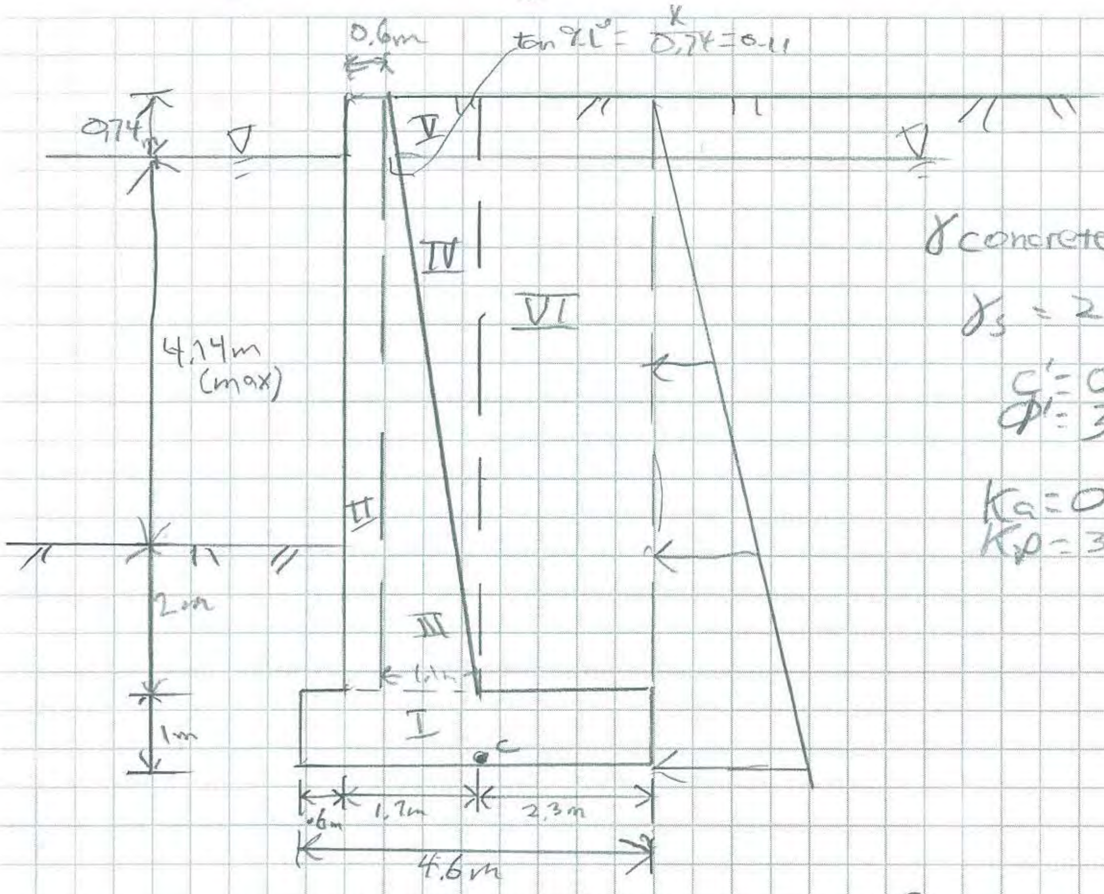
Client: USAID Job No.: Sheet 1 of 5

Description: Bridge 10 Wingwall Bearing Designed By: GL Date: 10-6-14

Checked By: Date:



γ concrete 23.6 Kn/m³

γ_s = 20.5 Kn/m³

c' = 0

ϕ' = 33°

K_a = 0.29

K_p = 3.39

Bearing length = 4.6m

Region	Area (m²)	γ (Kn/m³)	W (Kn/m)	Moment Arm (m)	M (Kn m/m) About point C
I	4.6	23.6	108.6	0	0
II	4.12	23.6	97.4	1.4	136.4
III	3.78	23.6	89.21	0.73	65.1
IV	3.04	20.5	62.32	0.66	41.1
V	0.77	20.5	15.85	0.55	8.7
VI	~~17.20~~ 15.82	20.5	~~352.68~~ 324.4	-1.15	~~-405.6~~ -373.1
F_{A1}	—	—	1.63	7.39	12.0
F_{A2}	—	—	~~86.4~~ 80.65	2.38	~~205.7~~ 191.9
F_{A3}	—	—	250.1	2.38	595.2
F_{A4}	—	—	31.4	3.57	112.1

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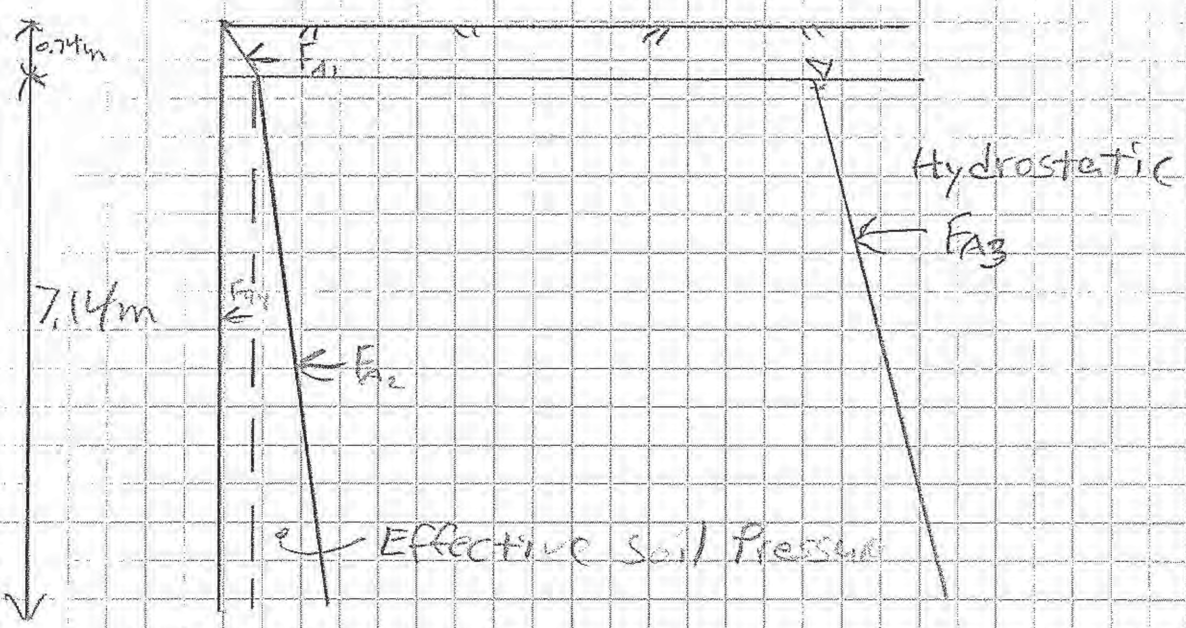
Client: USAID | Job No.: | Sheet 2 of 5

Description: Bearing Capacity ~~Abutme~~ Wingwall Bridge 10 | Designed By: GL | Date: 10-6-14

Checked By: | Date:

Calculation of Active Earth and Hydrostatic Pressure



$F_A = 0.5(\gamma)(H^2)(K_a)$

$F_{A1} = 0.5(20.5\ \frac{kn}{m^3})(0.74^2\ m)(0.29) = 1.63\ \frac{kn}{m}$

moment arm $= 7.14m + 1/3(0.74m) = 7.39\ \frac{kn}{m}$

$F_{A2} = F_{A1} + 0.5(20.5\ \frac{kn}{m^3} - 9.81\ \frac{kn}{m^3})(7.14^2\ m)(0.29) = \cancel{86.41}\ \frac{kn}{m}$ 80.65

moment arm F_{A2} : $1/3(7.14m) = 2.38\ m$

$F_{A3} = 0.5(9.81\ \frac{kn}{m^3})(7.14^2\ m) = 250.1\ \frac{kn}{m}$

moment arm $F_{A3} = F_{A2} = 2.38\ m$

$F_{A4} = (0.74m)(20.5\ \frac{kn}{m^3})(7.14m)(0.29) = 31.4\ \frac{kn}{m}$

moment arm $= 1/2(7.14m) = 3.57m$

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Client: USAID | Job No.: | Sheet 3 of 5
Description: Bearing Capacity Wingwall Bridge K0 | Designed By: GL | Date: 10-6-14
Checked By: | Date:

Eccentricity

$$e = \frac{136.4+65.1+41.1+8.7-\cancel{405.6}\;373.1+12.0+\cancel{205.7}\;191.9+595.2+112.1}{108.6+97.4+89.21+62.32+15.85+\cancel{352.68}\;324.4}$$

$$e = \frac{\cancel{770.7}\;789.4}{\cancel{726.1}\;697.8} = \cancel{1.06}\;1.13$$

$$\text{vertical stress } \sigma_v = \frac{\Sigma V}{B-2e} = \frac{\cancel{726.1}\;697.8}{4.6-2(\cancel{1.06}\;1.13)} = \cancel{292.8}\;298.2 \ \frac{kN}{m}$$

Soil Bearing Capacity

Assumptions:
- cohesionless soil
- effective stress analysis
- drained strength parameters
- groundwater at surface
- bottom of footing 2m below scour
- eccentric load = eccentric footing dimen
- service I load conditions

q_n = nominal bearing resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 B N_{\gamma m} C_{wq}$$

cohesion = 0 $\quad c = 0$

$$q_n = \gamma D_f N_{qm} C_{wq} + 0.5 B N_{\gamma m} C_{wq}$$

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Client: USAID Job No.: Sheet 4 of 5

Description: Bearing capacity Wingwall Bridge 10 Designed By: GL Date: 10-6-14

Checked By: Date:

$N_{qm} = N_q S_q d_q i_q$

$N_{\gamma m} = N_\gamma S_\gamma i_\gamma$

Where N_q = nominal bearing capacity for surcharge (table 10.6.3.1.2a-1)

$\phi' = 33°$ $N_q = 26.1$

N_γ bearing capacity soil unit weight (table 10.6.3.1.2a-1)

$N_\gamma = 35.2$

γ = total moist soil unit weight $= 20.5 \frac{kN}{m^3}$

d_q = footing embedment depth = 2 m

Eccentricity calculated previously = ~~1.06~~ 1.13

$B' = B - 2e$

$B' = 4.6m - 2(\cancel{1.06}\ 1.13) = \cancel{2.48}\ 2.34$

$L' = L = 4.6 m$

$A' = L'B' = 4.6m(\cancel{2.48m}\ 2.34) = \cancel{11.4}\ 10.8$

$C_{wq} = C_{w\gamma}$ = correction factor for groundwater depths

Table 10.6.3.1.2a-2 $C_{wq} = C_{w\gamma} = 0.5$

S_q = footing shape factor table 10.6.3.1.2a-3

$S_q = 1 + (\frac{B}{L} \tan\phi) = 1 + \frac{\cancel{2.48}\ 2.34}{4.6} \tan 33° = \cancel{1.35}\ 1.33$

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Client: USAID Job No.: Sheet 5 of 5

Description: Wing Wall Designed By: GL Date: 10-6-14

Bearing Capacity Bridge 10 Checked By: Date:

S_γ = footing shape factor table 10.6.3.1.2a-3

$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{\cancel{2.48}\ 2.34}{4.6}\right) = \cancel{0.78}\ 0.80$$

load indication factors = 1 = i_q and i_γ

d_q = correction factor to account for soil around footing

$d_q = 1$ conservative

⇒ Find N_{qm} and $N_{\gamma m}$

$$N_{qm} = N_q S_q d_q i_q = 26.1(\cancel{1.35}\ 1.33)(1)(1) = \cancel{35.24}\ 34.7$$

$$N_{\gamma m} = N_\gamma S_\gamma i_\gamma = 35.2(\cancel{0.78}\ 0.80)(1) = \cancel{27.46}\ 28.2$$

Find q_m

$$q_m = \gamma D_q N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$$

$$q_m = \left(20.5\tfrac{kn}{m^3}\right)(2m)(\cancel{35.24}\ 34.7)(0.5) + (0.5)\left(20.5\tfrac{kn}{m^3}\right)(\cancel{2.48}\ 2.34m)(0.5)(\cancel{27.46}\ 28.2)$$

$$q_m = \cancel{722.4}\ 711.4\ \tfrac{kn}{m^2} + \cancel{349.0}\ 338.2\ \tfrac{kn}{m^2} = \cancel{1{,}071.4}\ 1{,}049.6\ \tfrac{kn}{m^2}$$

$$q_r = \phi q_m \quad \phi = 0.45 \quad 0.45\left(\cancel{1{,}071.4}\ 1{,}049.6\ \tfrac{kn}{m^2}\right) = \cancel{482.1}\ 472.3$$

$$\cancel{482.1}\ 472.3 > \cancel{292.8}\ 298.2$$

$q_r > \sigma_v$ ⇒ OK

Bearing Resistance Pier

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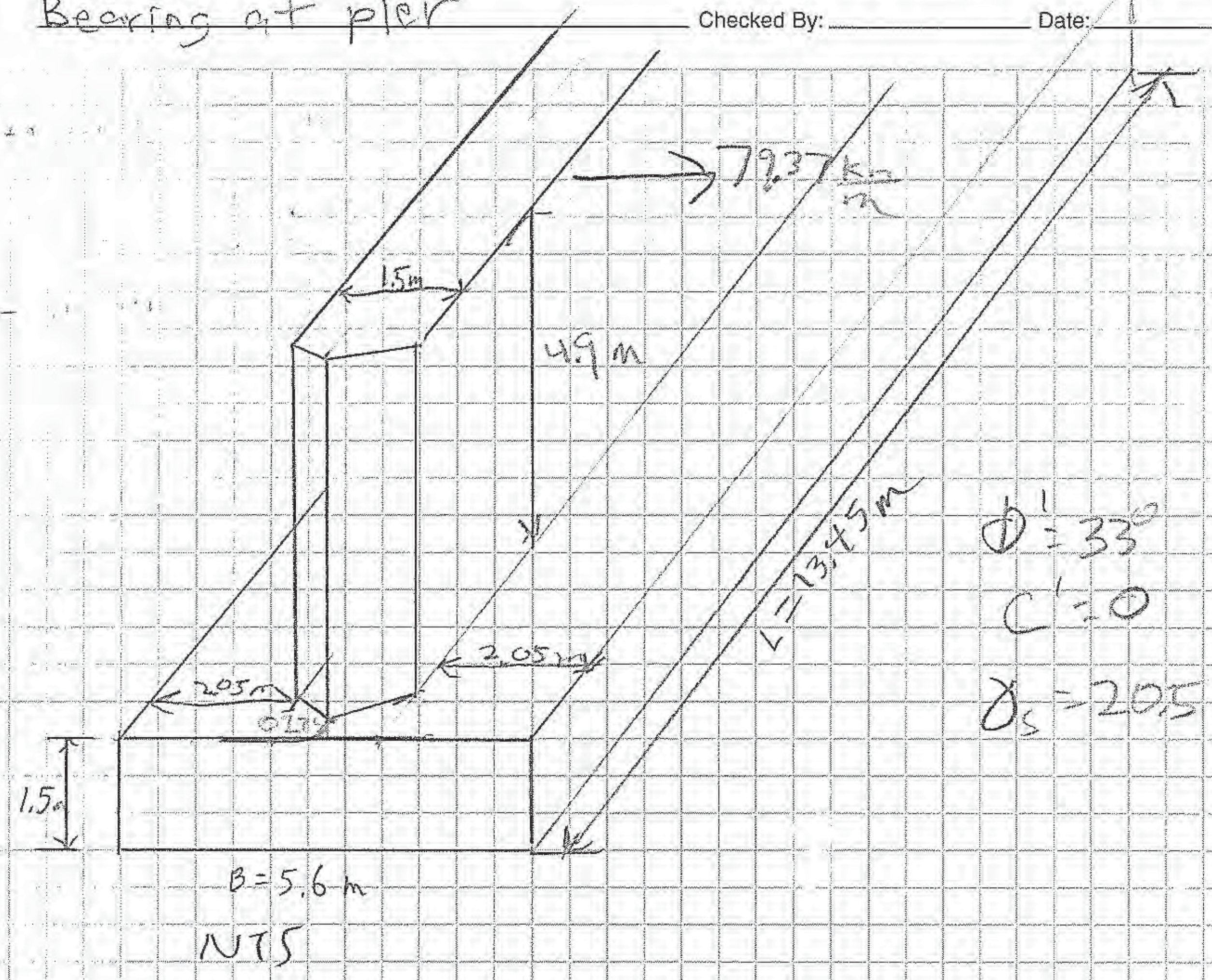
Client: USAID Job No.: Sheet 1 of 6

Description: Bridge 10 Designed By: GL Date: 10-6-14

Bearing at pier Checked By: Date:



$\phi' = 33°$

$C' = 0$

$\gamma_s = 20.5\ \frac{Kn}{m^2}$

Horizontal load $= 0.83\frac{K}{LF} + 0.85\frac{K}{LF} + 3.76\frac{K}{LF} = 5.44\frac{K}{LF}$

$5.44\frac{K}{LF} \times \frac{1000\,lbs}{K} \times \frac{1Kn}{224.8\,lb} \times \frac{3.28\,ft}{1\,m} = 79.37\frac{Kn}{m}$

PV $22.61\frac{K}{LF} + 9.87\frac{K}{LF} = 32.48\frac{K}{LF}$

$32.48\frac{K}{ft} \times \frac{1,000\,lbs}{K} \times \frac{1\,Kn}{224.8\,lb} \times \frac{3.28\,ft}{1\,m} = 473.91\frac{Kn}{m}$

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Client: USAID Job No.: Sheet 2 of 6

Description: Bridge 10 Designed By: GL Date: 10-6-14

Bearing at pier Checked By: Date:

Calculation per 2012 AASHTO LFRD

Bridge design specifications, section 10.6.3.1.2

Assumptions
- cohesionless soil
- effective stress analysis
- water table at surface
- bottom of footing 2m below scour
- eccentric load
- service I load conditions
- $\phi' = 33°$

Calculate q_r, factored resistance of soil

$$q_r = \phi_b q_n$$

ϕ_b = resistance factor per 10.5.5.2.2-1

in sand using spt $\phi_b = 0.45$

q_n = nominal bearing resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 B N_{\gamma m} C_{w\gamma}$$

cohesion = 0 $c = 0$

$$q_n = \gamma D_f N_{qm} C_{wq} + 0.5 B N_{\gamma m} C_{w\gamma}$$

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Client: USAID Job No.: Sheet 3 of 6
Description: Bridge 10 Designed By: GL Date: 10-6-14
Bearing Capacity at Pier Checked By: Date:

$N_{qm} = N_q S_q d_q i_q$

$N_{\gamma m} = N_\gamma S_\gamma i_\gamma$

where N_q = nominal bearing capacity for surcharge
table 10.6.3.1.2a-1 $\phi = 33^\circ$

$N_q = 26.1$

N_γ = bearing capacity factor for unit soil weight
table 10.6.3.1.2a-1

$N_\gamma = 35.2$

γ = total moist soil weight

$\gamma = 20.5 \frac{kn}{m^3}$

D_f = footing embedment depth = 2 m

Determine effective footing footing dimension
because loading is eccentric

$B' = B - 2e$

$e = \frac{M}{P} = \frac{(79.37 \frac{Kn}{m})(4.9m + 1.5m)}{473.9 \frac{kn}{m}} = 1.07m$

$B' = 5.6m - 2(1.07m) = 3.46m$

$L' = L = 13.45m$

$A' = B'L' = 3.46m \times 13.45m = 46.5 m^2$

$C_{wq} = C_{w\gamma}$ Correction factors for depth to water table assumed depth is 0

$C_{wq} = C_{w\gamma} = 0.5$

S_q – footing shape factor, table 10.6.3.1.2a-3

$$S_q = 1 + \left(\frac{B}{L}\tan\phi\right) = 1 + \left(\frac{3.46}{13.45}\tan 33\right) = 1.17$$

S_γ – footing shape factor

$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{3.46}{13.45}\right) = 0.90$$

i_q – load inclination factor

$$i_q = \left[1 - \frac{P_H}{P_V + cBL\cot\phi'}\right]^n$$

$$c = 0 \Rightarrow i_q = \left[1 - \frac{P_H}{P_V}\right]^n$$

$$n = \left[\left(2 + \frac{L}{B}\right)/\left(1 + \frac{L}{B}\right)\right]\cos^2\theta + \left[\left(2 + \frac{B}{L}\right)/\left(1 + \frac{B}{L}\right)\right]\sin^2\theta$$

θ is direction of horizontal load with respect to footing alignment

$\theta = 90°$

$\cos^2(90) = 0, \quad \sin^2 90 = 1$

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Client: USAID Job No.: Sheet 5 of 6

Description: Bearing capacity Pier Bridge 10 Designed By: GL Date: 10-7-14

Checked By: Date:

$$n = \left[\left(2+\frac{B}{L}\right)/\left(1+\frac{B}{L}\right)\right]$$

$$n = \left[\left(2+\frac{3.46}{13.45}\right)/\left(1+\frac{3.46}{13.45}\right)\right]$$

$$n = \frac{2.26}{1.26} = 1.79$$

$$i_q = \left[1-\frac{79.37 \text{ kN/m}}{473.91 \text{ kN/m}}\right]^{1.79} = (0.83)^{1.79} = 0.72$$

i_γ load inclination factor

$$i_\gamma = \left[1-\frac{P_H}{P_V + cBL\cot\phi}\right]^{n+1}$$ since $c=0$

$$i_\gamma = \left[1-\frac{P_H}{P_V}\right]^{n+1}$$

$$i_\gamma = \left[1-\frac{79.37 \text{ kN/m}}{473.91 \text{ kN/m}}\right]^{1+1.79} = 0.60$$

d_q correction factor for shearing resistance of soil above bearing elevation

$d_q = 1$ assume soil above bearing elevation does not provide shearing resistance

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Client: USAID Job No.: Sheet 6 of 6

Description: Bridge 10 Designed By: GL Date: 10-7-14

Bearing capacity pier Checked By: Date:

$N_{qm} = N_q S_q d_q i_q = (26.1)(1.17)(1)(0.72)$

$N_{qm} = 22.0$

$N_{\gamma m} = N_\gamma S_\gamma i_\gamma = (35.2)(0.90)(0.60)$

$N_{\gamma m} = 19.0$

q_n = nominal bearing resistance

$q_n = \gamma D_f N_{qm} C_{wq} + (0.5)\gamma B N_{\gamma m} C_{w\gamma}$

$q_n = (20.5 \frac{Kn}{m^3})(2m)(22.0)(0.5) + (0.5)(20.5 \frac{Kn}{m^3})(3.46m)(19.0)(0.5)$

$q_n = 451 \frac{Kn}{m^2} + 336.9 = 787.9 \frac{Kn}{m^2}$

$q_r = \phi_b q_n$ (factored bearing resistance)

$q_r = 0.45 (787.9 \frac{Kn}{m^2}) = 354.6 \frac{Kn}{m^2}$

Actual

$q_{actual} = \frac{P_V}{A'} = \frac{473.91 \frac{Kn}{m} \times 13.45m}{46.5 m^2} = 137.1 \frac{Kn}{m^2}$

$q_r > q_{actual}$

Section 3
Civil

Design Analysis

Discipline:	Civil	**Date:**	October 12, 2014

Design Submittal: Final Design Submittal
Site Location: Bridge #10
Prepared By: Tetra Tech

I. General Summary:

Bridge #10 was located on the Gardez to Khost Road in Afghanistan, spanning over a tributary immediately west of a main river. The existing Bridge #10 was destroyed by floods and a temporary pipe culvert was installed. A new bridge crossing was designed in 2010 (by Others) to increase the hydraulic capacity of the crossing. Prior to construction of the new bridge, USAID requested that Tetra Tech perform a topographical survey, geotechnical investigation, geotechnical analysis, hydraulic modeling and structural analysis in order to determine if the 2010 Design is in conformance with the latest AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (AASHTO LRFD) standards and adequate based on the complete hydraulic, geotechnical and structural analyses. Tetra Tech performed this work and submitted the "Scour Analysis and Foundation Study" to USAID (dated July 15, 2014). The study recommended that the Bridge #10 crossing be redesigned. This "Final Design" submittal completes the redesign.

The proposed Bridge #10 is a two-span structure, similar in design and detailing to Bridge #09. The proposed bridge superstructure and substructure shall be constructed out of reinforced concrete. Approach roadway work is required to transition from the existing roadway to the bridge. The proposed bridge includes a concrete scour mattress for protection against scour.

II. Basis of Design

- The roadway approaches will consist of asphaltic concrete pavement travel lanes and bituminous sealed shoulders. The roadway typical section is comprised of two 3.5 meter lanes with 1.0 meter shoulders with normal crown at the bridge and super elevated sections at the horizontal curves. There will be a transition to the bridge section which includes two 3.5m lanes with 0.5m shoulders and 1.2m sidewalks on each side.

- The horizontal alignment was designed to match into the recently constructed road both north and south of the bridge location. The road tangent through the proposed bridge was developed based on the alignment of the proposed river channel and the existing roadway north and south of the bridge.

- The vertical alignment was designed for a design speed of 50 km/h (30 mph) to match the existing roadway and meet the necessary proposed bridge deck elevation.

- Embankments adjacent to the river will be protected with rip rap stone.

- Stone masonry guardwalls are provided along the proposed roadway approaches to assist in guiding vehicles to the bridge crossing. The design of the guardwalls is not intended to be for crash attenuation.

- Grouted riprap slopes are used on the roadway embankments in areas where required for stability for embankment slopes in excess of 2:1 or as required for slope stability upstream of the bridge.

- A paved transition is provided at the side roads immediately north and south of the bridge to provide a smooth transition and minimize future maintenance at the intersection.

- Signage is provided for to alert traffic of the curved roadway. Additional signage has been provided to alert motorists of the side road intersections due to restricted sight distance of motorists crossing the bridge. Stop sign have been provided at the end of the side roads for safety.

III. Material Properties

- Approach roadway surface: Asphaltic concrete pavement conforming to specification section 32 12 16 to be obtained and manufactured locally.

- Approach roadway fill: Select fill conforming to specification section 31 20 00 intended to be obtained locally.

- Stone masonry walls conforming to specification section 32 32 40 and Rip Rap conforming to specification section 31 37 00: Stones intended to be obtained locally.

- Soil materials for roadway base courses and embankments conforming to specification section 31 20 00 shall be compacted to 95% maximum dry density as per ASTM D1557 or ASTM D4718, depending on fragment size. See technical Specifications for additional information.

- Reinforced Concrete pipe Culverts conforming to specification Section 33 46 20 to be obtained and manufactured locally.

IV. Code References

- US Army Corps of Engineers Afghanistan Engineer District AED Design Requirements: Vertical Curve Design and Superelevation Road Design March 2009
- AASHTO “A Policy on Geometric Design of Highways and Streets”, 6th Edition, 2011

- “Scour Analysis and Foundation Study” dated July 15, 2014 (prepared by Tetra Tech)

- Profiles and Plans for Gardez-Khost Rehabilitation Project (by Others)

Section 4
Structural

Design Analysis

Discipline:	Structural	**Date:**	October 12, 2014

Design Submittal: Final Design Submittal
Site Location: Bridge #10
Prepared By: Tetra Tech

I. General Summary:

Bridge #10 was located on the Gardez to Khost Road in Afghanistan, spanning over a tributary immediately west of a main river. The existing Bridge #10 was destroyed by floods and a temporary pipe culvert was installed. A new bridge crossing was designed in 2010 (by Others) to increase the hydraulic capacity of the crossing. Prior to construction of the new bridge, USAID requested that Tetra Tech perform a topographical survey, geotechnical investigation, geotechnical analysis, hydraulic modeling and structural analysis in order to determine if the 2010 Design is in conformance with the latest AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (AASHTO LRFD) standards and adequate based on the complete hydraulic, geotechnical and structural analyses. Tetra Tech performed this work and submitted the “Scour Analysis and Foundation Study” to USAID (dated July 15, 2014). The study recommended that the Bridge #10 crossing be redesigned. This “Final Design” submittal completes the redesign.

The proposed Bridge #10 is a two-span structure, similar in design and detailing to Bridge #09. The proposed bridge superstructure and substructure shall be constructed out of reinforced concrete. Approach roadway work is required to transition from the existing roadway to the bridge. The proposed bridge includes a concrete scour mattress for protection against scour.

II. Detailed Analysis:

The proposed two-span reinforced concrete bridge is comprised of 16.8 meter simple spans, with a total bridge length of 33.6 meters. The superstructure (girder, slab and barriers) and the substructure (abutments, retaining walls and piers) shall be reinforced concrete. The roadway is 4.0m wide in each direction of travel and has two 1.2 m wide sidewalks on each side of the roadway. The bridge is designed for an AASHTO LRFD HL-93 vehicle.

The substructure construction will potentially require dewatering and support-of-excavation in order to construct the proposed foundation.

If a crane is available during the superstructure construction, the beams can be precast offsite or on the approaches and placed using a crane. Using precast beams would accelerate the superstructure construction considerably. Since the reinforced concrete superstructure is heavy, deep girders are required to carry the load.

The superstructure, abutments, and piers have been designed to resist all applied loads as described in AASHTO “LRFD Bridge Design Specifications” 6th Edition, 2012. See the “Basis of Design” for design load information.

For related approach roadway work and limits of work, see the Civil section.

III. Basis of Design

- Dead Load: Selfweight of superstructure and substructure components
- Live Load: AASHTO LRFD HL-93 Vehicle
- Longitudinal Force: 5% of Live Load
- Wind Load: 2.44 kPa (50 psf) transverse
  0.59 kPa (12 psf) longitudinal
- Wind on Live Load: 148.8 kg/m (100 plf) transverse
  59.5 kg/m (40 plf) longitudinal
- Temperature Range Temperature Rise/Fall Range = 38.9 deg C (70 deg F)
- Seismic Load: Ss = 0.64g
  S1 = 0.47g
  SDC D
  PGA = 0.29g
- Hydraulic Data: (See Section 1 for additional information)
  - River bed elevation of 1816.370m at the upstream face of the bridge
  - River bed elevation of 1815.510 m at the downstream face of the bridge
  - Verifications that pier and abutment footings are below the scour line
  - Verification that the proposed bridge seat elevation has been set a minimum of 600mm above the 50-year flood elevation.
  - Hydraulic Data:
    - Design Flood Event = 50-yr
    - Design Velocity = 3.87 m/s
    - Design Water Surface Elevation =1818.54m
    - Scour Consideration:
      - As discussed in Section 1, based on the Hydraulic analysis, the scour depth at the abutments is 9.48m and the scour depth at the pier is 1.09m. Due to the deep scour cavities at the abutments, it is recommended that in lieu of providing scour countermeasures, the maximum top of footing elevation should be set at Elev. 1805.15m for all substructure units.
      - Since construction this deep is not practical or cost-effective, a reinforced scour mattress has been included below the bridge to protect the substructure elements from scour. The scour mattress is 200mm thick, sized to prevent uplift and to provide adequate thickness for an upper and lower mat of reinforcement.

- Geotech Data: (See Section 2 for additional information)
    - Groundwater level at channel grade
    - Weight of Soil = 20.5 kN/m^3 (130.4 pcf)
    - Angle of Internal Friction = 33 degrees
    - $K_o = 0.46$
    - $K_a = 0.29$
    - $K_p = 3.39$
    - Coefficient of Friction for Sliding = 0.57
    - Nominal Bearing Resistance:
        - As discussed in Section 2, the Nominal Bearing Resistance values for design were computed as:
            - 1874 kN/m^2 (39.1 ksf) for the abutments **
            - 1050 kN/m^2 (21.9 ksf) for the retaining walls **
            - 788 kN/m^2 (16.5 ksf) for the pier

        ** Since these values are much larger than typically used for design, the abutment and wall design has been based on Nominal Bearing Resistance value of 847 kN/m^2 (17.8 ksf) which is conservative.

- Load combinations are based on AASHTO "LRFD Bridge Design Specifications" 6th Edition, 2012.

IV. Material Properties

Concrete Properties:

- Concrete mix shall be ASTM C-150 Type 1 or Type 2 Portland Cement.
- $f'c = 27.5$ MPa (4000 psi)
- Reinforcement: $fy = 4218$ kg/cm^2 (60 ksi)

Anchor Bolts:

- ASTM F1554, Grade 36 (minimum) Steel

Soil Properties:

- As noted under Part II - Assumptions
- Compaction shall be 95% maximum dry density as per ASTM D1557 or ASTM D4718, depending on fragment size. See technical Specifications for additional information.
- The Contractor shall verify that the actual subsurface conditions meet the assumed geotechnical design parameters.

V. References

- AASHTO “LRFD Bridge Design Specifications” 6th Edition, 2012
- “Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Scour Analysis and Foundation Study” dated July 15, 2014 (prepared by Tetra Tech)
- “Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Geotechnical Report” dated June 17, 2014 (prepared by Tetra Tech)
- Detailed Engineering Design of Gardez-Khost Road Rehabilitation Project, dated June 2010 (prepared by Others)

VI. List of Attachments:

Calculations

Calculations

DESIGN CALCULATIONS
BRIDGE NO. 10
GARDEZ TO KHOST ROAD

TABLE OF CONTENTS

SUPERSTRUCTURE DESIGN

SEE BRIDGE #09 DESIGN ANALYSIS FOR CONSPAN MODEL, ANALYSIS & RESULTS

SUPERSTRUCTURE DESIGN

ELASTOMERIC BEARINGS DESIGN

Calculate Wind Loads ON superstructure & on Live Load

Wind on Superstructure W/S

Wind Pressures (AASHTO 3.8.1.2.2)

0.05 KSF, Transverse
0.012 KSF, Longitudnial

Superstructure Thickness

1500 mm Beam Height
225 mm Deck Thickness
1400 mm Sidewalk & Barrier
3125 mm = 10.25'

Length of Bridge = 16800 mm = 55.12'
Since bearing Pads without anchor bolts used at both ends of the bridge, the tributary are for wind is ½ the span

$\Rightarrow$ Tributary Area for Wind = $10.25' \times (\frac{1}{2} \cdot 55.12') = 282.5$ SF

Wind Force on Tributary Area;
Transverse $= 0.05 \times 282.5 = 14.1$ Kips $\Rightarrow \frac{14.1}{(6 \text{ bearings})} = 2.35$ K/brg.

Longitudinal $= 0.012 \times 282.5 = 3.39$ K $\Rightarrow \frac{3.39}{(6 \text{ brg.})} = 0.565$ K/brg.

Resultant $= \sqrt{2.35^2 + 0.565^2} = 2.42$ K/brg.

Wind On Live Load WL

Wind Pressures (AASHTO 3.8.1.3)

0.1 K/LF, Transverse
0.04 K/LF, Longitudinal

Wind Force, Transverse:
$0.1 \times 55.12'/2 = 2.76$ Kips $\Rightarrow \frac{2.76^K}{(6 \text{ brgs.})} = 0.46$ K/brg.

Wind Force, Longitudinal:
$0.04 \times 55.12/2 = 1.1$ Kips $\Rightarrow \frac{1.1^K}{(6 \text{ brgs.})} = 0.184$ K/brg.

Resultant $= \sqrt{0.46^2 + 0.184^2} = 0.5$ K/brg.

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB AESP Bridg 10
SHEET NO. ____ OF ____
CALCULATED BY SAM DATE 9/22/14
CHECKED BY APV DATE 10/7/14
SCALE ____

SEISMIC LOAD FROM SUPERSTRUCTURE

Since only expansion bearing used, the max seismic pull on the abutment is limited to the maximum shear capacity of the neoprene in the elestomeric bearing pad. The elastomeric bearing diameter is 15" with 60 durometer neoprene having a shear resistance of 75 psi (see attached reference).

Shear Resistance of Bearing $= \pi (7.5'')^2 (75\ PSI) = 13.25^K$

Force Per Linear Foot of Abutment Footing:

$= 13.25 \times (6\ \text{Bearings}) / 36.91' = 2.15\ ^K/_{L.F}$ of Abutment

Force Per L.F. of Pier $= 13.25^K \times (12\ \text{brgs.}) / 42.31' = 3.76\ ^K/_{L.F.}$ of Pier

LIVE LOAD BRAKING FORCE PER BEARING PAD

Load Per Linear Foot of Abutment:

$= 0.49\ ^K/_{LF}$ ← See "SUPERSTRUCTURE LOADING ON ABUTMENT – LATERAL FORCES" calculation spreadsheet.

Total Braking Load Per Abutment $= 0.49\ ^K/_{LF} \times 36.91' = 18$ Kips

Braking Load Per Bearing Pad $= \dfrac{18^K}{(6\ \text{Pad})} = 3^K/\text{Pad}$

TETRA TECH One Grant Street Framingham, MA 01701-9005 (508) 903-2000	JOB AESP Bridge 10 SHEET NO. ____ OF ____ CALCULATED BY SAM DATE 9/22/2014 CHECKED BY APV DATE 6/7/2014 SCALE ____

Table 2- 4 *Shear resistance values for NR and Neoprene 1973 AASHTO (19)*

Shear Resistance	Elastomer Type	Durometer
30psi (0.207MPa)	Natural Rubber	50
40psi (0.276MPa)	Natural Rubber	60
50psi (0.345MPa)	Natural Rubber	70
50psi (0.345MPa)	Neoprene	50
75psi (0.517MPa)	Neoprene	60
110psi (0.759MPa)	Neoprene	70

FOR REFERENCE ONLY

Table 2- 5 *Dimension tolerances for elastomeric bearings 1977 and 1983 AASHTO (20, 21)*

1) Overall Vertical Dimensions Average Total Thickness 1 ¼" (31.8mm) or less Average Total Thickness over 1 ¼" (31.8mm)	 -0, +1/8in. (3mm) -0, +1/4in. (6mm)
(2) Overall Horizontal Dimension 36in. (914mm) and less over 36in. (914mm)	 -0, +1/4in. (6mm) -0, +1/2in. (6mm) 12th edition -0, +1/4in. (6mm) 13th edition
(3) Thickness of Individual Layers of Elastomer (Laminated Bearing)	 ±1/8in. (3mm)
(4) Variation from a Plane Parallel to the Theoretical Surface (as determined by measurements at Top Sides Individual Nonelastic Laminates	 1/8in. (3mm) 1/4in. (6mm) 1/8in. (3mm)
(5) Position of Exposed Connection Members	1/8in. (3mm)
(6) Edge Cover of Embedded Laminates or Connection Members	 -0, +1/8in. (3mm)
(7) Size of Holes, Slots, or Inserts	±1/8in. (3mm)
(8) Position of Holes, Slots, or Inserts	±1/8in. (3mm)

Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: APL
Date: 10/9/2014

References:
American Association of State Highway and Transportation Officals (AASHTO) LRFD Bridge Design Specification, 2012
ASSHTO 14.7.6 - Elastomeric Pads and Steel-Reinforced Elastomeric Bearings - Method A

Notes:
In accordance with ASSHTO 14.7.6. - Elastomeric Pads and Steel-Reinforced Elastomeric Bearings - Method A
For Service I Limit State.
Unless otherwise noted, the resistance factor, Ø, shall be taken as 1.0.
Dynamic Load allowance shall not be included.

BEARING PAD GEOMETRY & QUANTITY

Type of Pad: Steel-Reinforced Bearings

Shape of Pad: Circular

Steel Reinforcment: 11 gage

	Support 1	Support 2		
	Left Support	Right Support		
Horizontal Fixity:	Expansion	Expansion		
Diameter, D :	15.000	15.000	in	
Elastomer Top Cover Thickness, h_{r_top} :	0.250	0.250	in	>= 0.25, OK
Elastomer Internal Layer Thickness, h_{r_int} :	0.375	0.375	in	
Elastomer Bottom Cover Thickness, h_{r_bot} :	0.250	0.250	in	>= 0.25, OK
Steel Reinforcement Thickness, h_{reinf} :	0.1196	0.1196	in	
Number of Steel Reinforcement Layers, Ns:	6	6		
Number of Internal Elastomeric Layers, Ne:	5	5		
Design Total Elastomer Thickness, h_{rt} :	2.375	2.375	in	
Design Total Steel Reinforcement Thickness, h_s :	0.718	0.718	in	
Effective Bearing Thickness :	3.093	3.093	in	
Actual Total Bearing Thickness :	3.093	3.093	in	
Req't --> Top Elast Layer < 70% of Inner Layer :	OK	OK		
Req't --> Bot Elast Layer < 70% of Inner Layer :	OK	OK		
Number of Beams, Nbeams :	6			
Number of Bearings per Beam, Nb_beam :	2			
Number of Bearings at Each Beam End, Nb_beam end :	1	1		
Total Number of Bearing per Abutment, Nb_Abut :	6	6		

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Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: AR
Date: 10/9/2014

SHAPE FACTOR (S) - AASHTO 14.7.5.3.3 & 14.7.6.3.3

Shape Factor, S for Circular Bearing Pads

	Abutment 1			Abutment 2				
	Internal	Top Cover	Bot Cover	Internal	Top Cover	Bot Cover		
Diameter, D :	15.00	15.00	15.00	15.00	15.00	15.00		
Thickness of Elastomer Layer, h_r :	0.375	0.250	0.250	0.375	0.250	0.250		
Shape Factor, S :	10.000	15.000	15.000	10.000	15.000	15.000	$= D / (4 * h_{ri})$	AASHTO 14.7.5.1-2

MATERIAL PROPERTIES

F_y: 36.0 ksi
F_{sr}: 24.0 ksi

Grade: 3.0
Nominal Hardness : 60 durometer
Shear Modulus @ 73F, G_{min}: 80.0 psi
Shear Modulus @ 73F, G_{max}: 175.0 psi

LRFD SERVICE I LOAD COMBINATION LIMIT STATE

LOAD PER BEAM END	* Load Factor	F_v (Kips)	$F_{h\text{-}long}$ (Kips)	
DC	1.00	76.70		
DW	1.00	0.00		<=== For the purpose of design, DW and DC will be lumped into DC
LL+IM+PL+BR+LS	1.00	79.90	3.00	<=== See backup hand calcs. for braking horizontal force
WS	0.30		2.42	<=== See backup hand calculations
WL	1.00		0.50	
TU	1.20			<=== See calcs. Below
SUM		**156.60**	**5.92**	

*See AASHTO Table 3.4.1-1 for LRFD Load Factors for Service I Limit State

Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: APL
Date: 10/9/2014

COMPRESSIVE STRESS - INTERNAL LAYER - AASHTO 14.7.6.3.2

Typical Bearing Pad Plan Area, A_1:	176.71	in^2	
h_r_max:	0.375	in	<-- hr_max = hr_int
S:	10.00		<-- S = S_int

Service I Limit State

Avg. Compressive Stress due toTotal Load, σ_s:	0.886	ksi = ΣF_v / A_1/Nb_beam end	σ_s < =	1.25*Gmin*Si	1.000	ksi	OK	AASHTO 14.7.6.3.2-7
			σ_s < =		1.250	ksi	OK	AASHTO 14.7.6.3.2-8

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Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: APL
Date: 10/9/2014

COMPRESSIVE DEFLECTION - AASHTO 14.7.5.3.3 & 14.7.6.3.3

Average Compressive Stress (DL), σ_{DL} = 0.434 ksi = ΣF_v (DL) / A_1 / Nb_beam end
Average Compressive Stress (LL), σ_{LL} = 0.452 ksi = ΣF_v (LL) / A_1 / Nb_beam end

Creep Deflection at 25 Years, a_{cr}= 0.35 — AASHTO Table 14.7.6.2-1

	Support 1			Support 2					
	Internal	Top Cover	Bot Cover	Internal	Top Cover	Bot Cover			
S :	10.00	15.00	15.00	10.00	15.00	15.00		$= D / (4 * h_{ri})$	AASHTO 14.7.5.1-2
Effective Modulus of B.P. in Compression, E_c :	84	189	189	84	189	189	ksi	$= 4.8 * G_{min} * S^2$	AASHTO C14.6.3.2-1
Computed Compressive Strain (DL), ε_{DL} :	0.00517	0.00230	0.00230	0.00517	0.00230	0.00230	ksi	$= \sigma_{DL} / E_c$	AASHTO 14.7.6.3.3-1
Computed Compressive Strain (LL), ε_{LL} :	0.00538	0.00239	0.00239	0.00538	0.00239	0.00239	ksi	$= \sigma_{LL} / E_c$	AASHTO 14.7.6.3.3-1
Deflection (DL), δ_{DL} :		0.01084			0.01084		in	$= \Sigma (\varepsilon_{DL} * h_r)$	AASHTO 14.7.5.3.6-1
Deflection (LL), δ_{LL} :		0.01129			0.01129		in	$= \Sigma (\varepsilon_{LL} * h_r)$	AASHTO 14.7.5.3.6-2
a_{cr} * Deflection (DL) :		0.00379			0.00379		in	$= a_{cr} * \delta_{DL}$	
Long Term Dead Load Deflection, δ_{lt} :		0.01463			0.01463		in	$= \delta_{DL} + (a_{cr} * \delta_{DL})$	AASHTO 14.7.5.3.6-3

Check Initial Deflection — AASHTO 14.7.6.3.3

	Support 1			Support 2				
	Internal	Top	Bottom	Internal	Top	Bottom		
h_r :	0.3750	0.2500	0.2500	0.3750	0.2500	0.2500	in	
$0.09 * h_r$:	0.0338	0.0225	0.0225	0.0338	0.0225	0.0225	in	
$\delta_{DL} + \delta_{LL}$:	0.0040	0.0012	0.0012	0.0040	0.0012	0.0012	in	$= (\varepsilon_{DL} * h_r) + (\varepsilon_{LL} * h_r)$
Check Initial Deflection :	OK	OK	OK	OK	OK	OK		

Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: AP
Date: 10/9/2014

SHEAR - AASHTO 14.7.5.3.2 & 14.7.6.3.4

Girder Material: Concrete

Span Length, L : 55.120 ft
Superstructure width, W : 35.930 ft

Longitudinal Dist. Subject to Temp., L_T : 27.560 ft/brg
Transverse Dist. Subject to Temp., W_T : 17.965 ft/brg

Thermal Movement

Expansion

Thermal Expansion, α:	0.000006	in / in / °F
$\Delta_T = T_{rise}$:	70	°F (conservative)
Long. Thermal Movement, δ_{TL} :	0.139	in
Trans. Thermal Movement, δ_{TT} :	0.091	in
Resultant Thermal Movement, δ_T :	0.166	in
Load due to Temp, $H_{u\text{-rise}}$:	2.159	k

Contraction

Thermal Expansion, α:	0.0000060	in / in / °F
$\Delta_T = T_{fall}$:	70	°F (conservative)
Long. Thermal Movement, δ_{TL} :	0.139	in
Trans. Thermal Movement, δ_{TT} :	0.091	in
Resultant Thermal Movement, δ_T :	0.166	in
Load due to Temp, $H_{u\text{-fall}}$:	2.159	k

$\delta_{TL} = L_T * \alpha * \Delta_T$
$\delta_{TT} = W_T * \alpha * \Delta_T$

$H_u = G_{max} * A_1 * \delta / h_{rt}$

Controlling

Long. Thermal Movement, δ_{TL} :	0.139	in
Trans. Thermal Movement, δ_{TT} :	0.091	in
Resultant Thermal Movement, δ_T :	0.166	in
Load due to Temp., H_u :	2.159	kips

Total Thermal Force for Abutment Design : 12.95 kips = H_u * Number of Bearings per Abutment

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Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: AR
Date: 10/9/2014

SHEAR - AASHTO 14.7.5.3.2 & 14.7.6.3.4 - CONTINUED

Shrinkage

Coeff of Shrinkage, α_s:	0.000200			AASHTO 5.4.2.3
Long. Shrinkage Movement, δ_{SL}:	0.066	in	$\delta_{SL} = L_T * \alpha_s$	
Transv. Shrinkage Movement, δ_{ST}:	0.043	in	$\delta_{ST} = W_T * \alpha_s$	
Resultant Shrinkage Movement, δ_S:	0.079	in		

Wind and Breaking Loads — Since there are no fixed supports, these Wind and Breaking forces will contribute to the bearing horiz. deformation

Load due to Wind and Breaking, H_u:	4.226	kips	
Long. Wind and Breaking Movement, δ_{LRL}:	0.710	in	δ_LRL = Hu * hrt / Gmin / A1

Total Movement

Total Long. Movement, δ_{LT}:	0.915	in	$\delta_{TL} + \delta_{SL} + \delta_{LRL}$
Total Trans. Movement, δ_{TT}:	0.134	in	$\delta_{TT} + \delta_{ST}$
Resultant Movement, δ_{Total}:	0.925	in	

Δs: 0.955 in — Δs = Factored $\delta = \delta * \gamma_{TU}$ — AASHTO 14.6.3.1-2

2*Δs:	1.910	in	<= Suprt. 1, h_{rt}:	2.375	in	<-- OK	AASHTO 14.7.6.3.4-1
			<= Suprt. 2, h_{rt}:	2.375	in	<-- OK	

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Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: APL
Date: 10/9/2014

ROTATION - AASHTO 14.7.6.3.5

Rotation Requirement: S_i^2 / n < 22 AASHTO 14.7.6.1

Shape Factor of an elastomeric bearing, S_i:	10.00			
Number of interior layers of elastomer, n:	5.00			
S_i^2 / n:	20.00	<	22	OK

STABILITY - AASHTO 14.7.6.3.6

Support	Total Bearing Thickness	< =	D/4	Req't Met
1	3.093	< =	3.75	**OK**
2	3.093	< =	3.75	**OK**

Circular Steel-Reinforced Elastomeric Bearing Design

GENERAL INFORMATION

Project Number: Gardez Khost Road Construction of Bridge 10
Description: Elastomeric Bearing Pad Desing
Structure: Span 1, Beam Bearings at Abutment and Pier (By inspection, Interior beams control)

Designed By: SAM
Checked By: APL
Date: 10/9/2014

REINFORCEMENT - AASHTO 14.7.6.3.7 & 14.7.5.3.5

Service Limit State

ASSHTO 14.7.5.3.5-1

Avg Comp Stress due to Total Load, σ_s = 0.886 ksi
Yield Stength of Steel Reinforcement, F_y = 36.0 ksi

Support	h_{max} in	$(3 * h_{max} * \sigma_s) / F_y$	< =	h_s in	Req't Met
1	0.375	0.0277	< =	0.120	OK
2	0.375	0.0277	< =	0.120	OK

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SUBSTRUCTURE DESIGN

SUBSTRUCTURE DESIGN

BACKUP INFORMATION FOR SUBSTRUCTURE DESIGN

* VERTICAL GEOMETRY

*THERMAL LOADS

*SEISMIC LOADS AND RESTRAINTS

VERTICAL GEOMETRY – ABUTMENTS & PIER

	T.O. ROADWAY	BEAM SEAT ELEV. BM 3&4	BM 2&5	BM 1&6
℄ BRG N. ABUT	1821.832	1819.960	1819.923	1819.886
℄ PIER	1821.748	1819.876	1819.839	1819.802
℄ BRG S. ABUT	1821.664	1819.792	1819.755	1819.718

STRUCTURE DEPTH
W.S 50 mm
DECK 225 mm
BM 1500 mm
BRG 78.5 mm
$\Sigma = 1.8535$ m

ΔEL DUE TO CROSS-SLOPE
BM 3/4 = .02 x .925 = .0185 m
Δ = 37 mm
BM 2/5 = .02 x (.925 + 1.85) = .0555 m
Δ = 37 mm
BM 1/6 = .02 x (.925 + 1.85 + 1.85) = .0925 m

TOP OF CURTAIN WALL = 1.500 m ABOVE MIN BRIDGE SEAT
NORTH = 1819.886 + 1.5 = 1821.386 m
PIER = 1819.802 + 1.5 = 1821.302 m
SOUTH = 1819.718 + 1.5 = 1821.218 m

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SHEET NO. ____ OF ____
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VERTICAL GEOMETRY – RETAINING WALLS

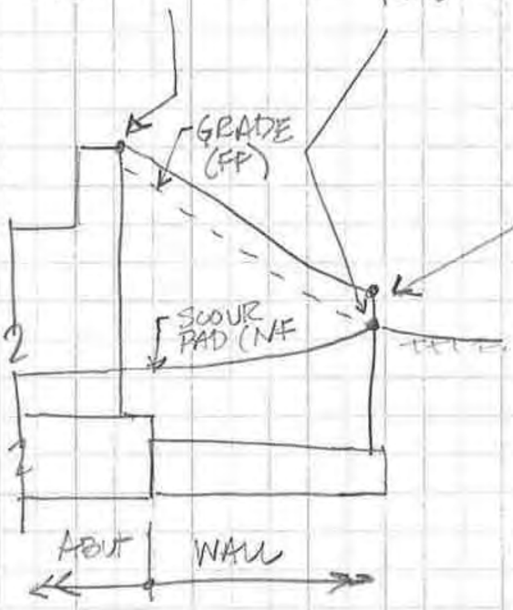
TOP OF WALL @ ABUT = TOP OF CURTAIN WALL

TOP OF WALL @ LOW POINT = 150mm ABOVE GRADE

BOT OF WALL FOOTING = BOT OF ABUT FTG = ~~1813.500m~~ 1812.5

UPDATED PER FINAL CIVIL

WALL	T.O. CURTAIN WALL	GRADE @ END	LOW T.O.W	WALL HEIGHT MIN–MAX
NW (6.0m)	1821.386 m	1818.250 +/-	1818.450	~~5950–7886~~ 4950 8886
NE (6.5m)	1821.386 m	1817.900 +/-	1818.050	~~4550–7886~~ 5550 8886
SE (7.0m)	1821.218 m	1817.200 +/-	~~1817.350~~ *1818.150	~~3850–7718~~ *1050 5650 8718
SW (6.0m)	1821.218 m	1818.200 +/-	1818.350	~~4850–7718~~ 5856 8718



WALL DESIGN FOR SECTION 30% AWAY FROM TALLEST HEIGHT OF WALL

@ NW: H = 7.25m
H_{STEM} 6.25m
H_{ftg} = 1.0m

GOVERNS

* REQUIRED FOR CONCRETE APRON

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ABUTMENT DESIGN - THERMAL LOADS

THERMAL MOVEMENT

$\Delta T \approx 70°F$ DEPENDING ON WHEN CONSTRUCTION OCCURS

$L_{LONG} = 55.12$ FT $L_{LAT} = 17.97$ ft

$\alpha = 6.0 \times 10^{-6} / °F$ (PER AASHTO C5.4.2.2)

$\delta_T = L \alpha \Delta T$ δ_T LONG $= 0.023' = 0.28"$

δ_T LAT $= 0.008' = 0.091$

THERMAL FORCE DUE TO DEFORMATION

$H = GA \frac{\Delta}{h_{rt}}$ per AASHTO EQ 14.6.3.1-2

$G = 175$ PSI MAX PER AASHTO 14.7.5.2

$A = \cancel{254.5} in^2$ $(176 in^2)$

$\Delta_T = \sqrt{0.28^2 + 0.091^2} = 0.3"$

$h_{rt} = 3"$

$H = \frac{.175 (\cancel{254.5})^{176} (0.3)}{3} = \cancel{4.5}^{3.1}$ KIPS (3.08^k)

$F_T = H \times \#$ BEARINGS $= \cancel{4.5}^{3.1}(6) = \cancel{27}^{18.6}$ KIPS / 2 Abut. $= \cancel{13.5}^k$ / Abut (9.3)

UNIFORM LOAD FOR ABUTMENT DESIGN

$W_T = \frac{\cancel{13.5}^{9.3} \text{ KIPS}}{37'} = \cancel{0.37}$ KLF (0.25)

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Expansion Joint Opening Size Calculation

Expansion Joint Shown On drawing Assumed at 20°C = 68°F

Temperature Rise and fall assumed 70° ← conservative

Change In Gap Width $\delta = L \alpha \Delta T$

$L = 55.12'$

$\alpha = 6 \times 10^{-6} / °F$

$\Delta T = 70 °F$

$\Rightarrow \delta = 55.12' \times 6 \times 10^{-6} \times 70 = 0.023' = 0.28'' = 7.1 \text{ mm}$

Use 8 mm ✓

Gap Adjustment per 10°c (18°F)

$\delta_{10°c} = 55.12' \times 6 \times 10^{-6} \times 18°F = 0.00595' = 0.0714'' = 1.81 \text{ mm}/10°c$

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SEISMIC RESTRAINT DESIGN

FROM SUPERSTRUCTURE DEAD LOAD CALCS, THE DEAD LOAD OF ONE SPAN IS:

DC = 922 KIPS
DW = 43 KIPS
DL = 965 KIPS

- LONGITUDINAL

IN THE LONGITUDINAL DIRECTION, THE EQ LOAD FROM 2 SPANS SHALL BE RESTRAINED BY 1 ABUTMENT BACKWALL. THIS ASSUMES THAT THE ELASTOMERIC BEARINGS HAVE FAILED AND THE "FLOATING" BRIDGE SUPERSTRUCTURE SLAMS INTO THE BACKWALLS.

PER AASHTO 3.10.9.5, EQ LOAD = DL × As

$$EQ = \frac{965\,(2\text{ SPANS})}{37'\text{ ABUT}} \times 0.29 = 15.13\text{ KLF}$$

THIS LOAD SHOULD BE RESISTED BY THE BACKWALL BY SHEAR FRICTION PER AASHTO 5.8.4.

$$V_n = cA_{cv} + \mu(A_v f_y + P_c)$$

$A_{c_1} = 450\text{mm} \times 1\text{ft} = 17.7" \times 12" = 212.52\text{ IN}^2$

$c = 0.28$ ksi
$\mu = 1.0$
$K_1 = 0.3$
$K_2 = 1.8$

} AASHTO 5.8.4.3, BASED ON CONSTRUCTION JOINT AT BASE OF BACKWALL

$$A_v = 16\phi @ 300\text{ EF} = 0.31\text{ IN}^2(2)\left(\frac{12'}{11.8"}\right) = 0.63\text{ IN}^2/\text{FT}$$

$P_c = 0$ (CONSERVATIVE)

$$V_n = 0.28(212.52) + 1.0(0.63 \times 60) = 59.5^K + 37.8^K = 97.3^{\text{KIPS}}$$

$V_r = \phi V_n$ $\phi = 1.0$ FOR EXTREME EVENT $\therefore V_r = 97\text{ KLF} > 15\text{ KLF}$

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SEISMIC RESTRAINT DESIGN (CONTINUED)

- LONGITUDINAL

THEREFORE, THE SHEAR FRICTION OF A 450mm THICK BACKWALL REINFORCED WITH 16φ@300mm EACH FACE IS ADEQUATE. THIS APPROACH IS ADEQUATE, AS IT IGNORES THE RESISTANCE DUE TO THE PASSIVE PRESSURE BEHIND THE BACKWALL IN A SEISMIC EVENT.

CHECK FLEXURAL CAPACITY OF BACKWALL REBAR

$$a = \frac{A_s F_y}{.85 f'_c b} = \frac{0.31(60)}{.85(4)(12)} = 0.456''$$

$$M_n = A_s F_y \left(d - \frac{a}{2}\right) \qquad d = 450 - 50 - 8 = 392\text{ mm} = 15.4''$$

$$M_n = 0.31(60)\left(15.4 - \frac{0.456}{2}\right) = 282\text{ K·IN}$$

$$\phi M_n = .9(282) = 254\text{ K·IN} = 21.2\text{ K·FT}$$

COMPUTE CG SUPERSTRUCTURE FROM SUPERSTRUCTURE LOAD CALCS

DIST ABOVE BRIDGE SEAT

	W (KIPS)	Y	WY
BEAMS	480	2.72'	1306
SW	59	6.41'	378
BARRIER	68	8.21	558
WS	43	6.0	258
DIAPH	89	3.09'	275
DECK	225	5.54	1247
Σ	964		4022

$$\therefore \bar{y} = \frac{4022}{964} = 4.17'$$

THIS IS AT THE TOP OF THE BACKWALL & IS OVERLY CONSERVATIVE.

∴ APPLY AT MIDHEIGHT OF BACKWALL = $\frac{1500 + 785}{}$ = 789 mm = 2.6 ft

M_U = 15.13 KLF × 2.6' = 39.3 $\frac{\text{K·FT}}{\text{FT}}$ > φMn ~~OK~~

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	SHEET NO. 2 OF
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	CHECKED BY SAM DATE 9/30/14
	SCALE

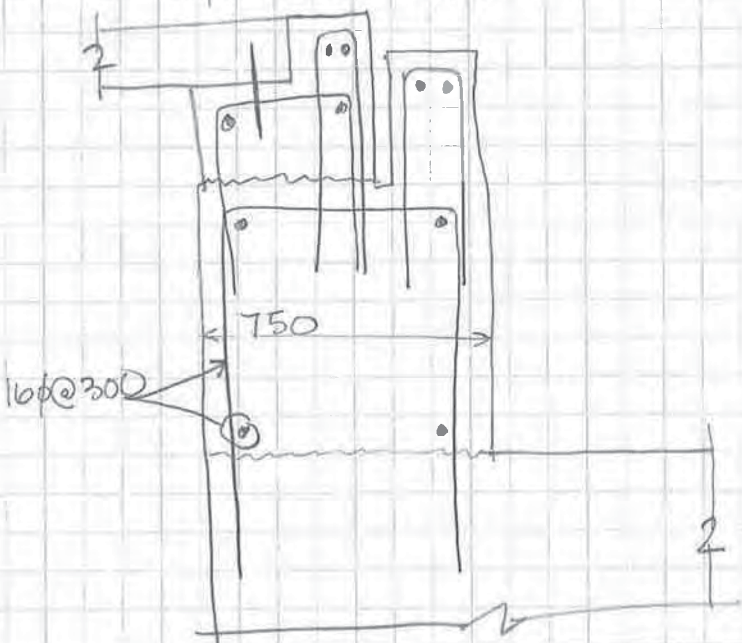
SEISMIC RESTRAINT DESIGN (CONTINUED)

- LONGITUDINAL

ALTHOUGH THERE IS ADEQUATE CAPACITY, IT IS CLEAR THAT ADDITIONAL CAPACITY WILL BE REQUIRED FOR BRIDGE 9 SINCE BRIDGE 9 HAS 3 SPANS.

RECOMMENDATION:

SINCE THE BACKWALL REINF IS SO CONGESTED, SIMPLIFY OVERALL REINF BY INCREASING THICKNESS TO ELIMINATE BUMP-OUT FOR APPROACH SLAB



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SEISMIC RESTRAINT DESIGN (CONTINUED)

- TRANSVERSE

IN THE TRANSVERSE DIRECTION, EACH END OF EACH SPAN IS RESTRAINED BY THE KEEPER BLOCK AND CHEEK WALL.

$$EQ = \frac{965 \text{ KIPS}}{2 \text{ ENDS}} \times 0.29 = 140 \text{ KIPS}$$

CHEEKWALL $\left.\begin{array}{l} w = 450\text{mm} \\ L = 1700 \text{ mm} \end{array}\right\}$ @ ABUTS w/ 20 ϕ @ 150

$$A = 1.48' \times 5.58' = 8.2 \text{SF}$$

CONSIDER SHEAR FRICTION

- Abut $A_c = 8.2$ SF   22- #6 BARS = 8.8 IN^2

$$\phi V_{n\,abut} = 1.0(0.28)(8.2)(144) + (8.8)(60) = 330 + 528 = 858^{\text{KIPS}}$$

$$\phi V_n > EQ = 140^{\text{KIPS}}, \underline{OK}$$

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SEISMIC RESTRAINT DESIGN (CONTINUED)

- TRANSVERSE (PIER)

PIER HAS 20φ@150 AROUND THE PERIMETER WITH 16φ@200 CONFINEMENT

EACH PIER CHEEKWALL RESTRAINS 2 SPANS = $140^K \times 2 = 280^{KIPS}$

TO BE CONSERVATIVE, CONSIDER THE FOLLOWING

1500 mm ≈ 4.9'
= 58.8" → # BARS = $\frac{(58.8"-4")}{6}$ ⇒ 10

$A_c = 4.9 \times 2.54 = 12.45$ SF

$525 + 750/3 = 775$ mm = 2.54' = 30.5"

↳ # BARS = $\frac{(30.5-4)}{6}$ ⇒ 5

$\therefore A_V = (10+5) \times 2 \times 0.44 = 13.2 \text{ IN}^2$

$V_N = cA_{cv} + \mu(A_v f_y + P_c)$
$= 0.28(12.45)(144) + 1.0(13.2)(60) = 502 + 792 =$ 1294 KIPS

$\phi V_n = 1.0 V_n = 1294$ KIPS $> V_u$ OK

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SHEET NO. 5 OF
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SCALE

SEISMIC RESTRAINT DESIGN (CONTINUED)

– TRANSVERSE

CHECK FLEXURAL CAPACITY

$M_U = 140^K \times 2.6' = 364 \text{ K·FT}$

ARM, SEE LONGITUDINAL CALCS FOR BACK-UP

ABUTMENT

$a = \frac{A_s F_y}{.85(f'c)(b)} = \frac{0.88(60)}{.85(4)(12)} = 1.29$ (PER FT)

$d = 450\text{mm} - 50 - 10 = 390\text{mm} = 15.35''$

$\phi M_n = \phi A_s F_y(d - a/2) = .9(.88)(60)(15.35 - 1.29/2)$

$\phi M_N = 699 \frac{\text{K·IN}}{\text{FT}} = 58.2 \frac{\text{K·FT}}{\text{FT}} \times 5.58' = 325 \text{ K·FT}$

(89% OF REQ'D)

1700MM

TRY #7@6" $A_s = 0.6(2) = 1.20 \text{ IN}^2$

$a = 1.76''$ $d = 15.3''$ $\phi M_N = 78 \frac{\text{K·FT}}{\text{FT}} \times 5.58'$

$\phi M_N = 436 \text{ K·FT} > 364$ OK

PIER

BASED ON THE 30.5" × 58.8" "EQUIVALENT" SECTION

$a = 1.29''$ (PER FT)

$d = 30.5 - 2'' - 0.375'' = 28.1''$

$\phi M_N = .9(.88)(60)(28.1 - 1.29/2) = 1305 \text{ K·IN} = 108.7 \text{ K·FT/FT}$

$\phi M_N = 108.7 \frac{\text{K·FT}}{\text{FT}} \times \frac{58.8''}{12} = 533 \text{ K·FT} < 364 \times 2 = 728 \text{ K·FT}$

(73% OF CAPACITY)

TETRA TECH One Grant Street Framingham, MA 01701-9005 (508) 903-2000	JOB: HESP BRIDGE 10 SHEET NO.: 6 OF: CALCULATED BY: APL DATE: 9/18/14 CHECKED BY: SAM DATE: 9/30/14 SCALE:

SEISMIC RESTRAINT DESIGN (CONTINUED)

- TRANSVERSE (PIER)

SIMILAR TO ABUT, TRY #7@6"

$a = 1.76"$

$d = 28"$

$\phi M_N = .9(1.2)(60)(28-1.76/2)/12 = 146\ K \cdot FT \times \frac{58.8'}{12}$

$\phi M_N = 717\ K \cdot FT \approx M_U = 728\ K \cdot FT$

(SAY OK, THIS SHOWS CAPACITY IS 99% OF REQUIRED, AND IS BASED ON THE CONSERVATIVE ASSUMPTION OF USING ONLY 1/3 OF THE CONE. IN REALITY, THE DEPTH OF THE CHEEKWALL IS GREATER WHICH WILL INCREASE CAPACITY.)

RECOMMENDATION:

CURTAIN WALL @ ABUT - MAINTAIN 450mm THICKNESS
INCREASE TO 22φ @ 150

CURTAIN WALL @ PIER - MAINTAIN 525mm + 750 CONE
INCREASE TO 22φ @ 150

TETRA TECH One Grant Street Framingham, MA 01701-9005 (508) 903-2000	JOB AESP BRIDGE 10
	SHEET NO. 7 OF
	CALCULATED BY APL DATE 9/25
	CHECKED BY SAIM DATE 9/30/14
	SCALE

SEISMIC RESTRAINT DESIGN — BRIDGE 9

- TRANSVERSE

BASED ON THE BRIDGE 10 DESIGN. THE LATERAL LOADS ON THE CHEEKWALL ARE THE SAME.

ABUT - USE 450mm w/ 22φ @ 150
PIER - USE 525mm + 750 mm CONE w/ 22φ @ 150

- LONGITUDINAL

SIMILAR TO BRIDGE 10, BUT ONE BACKWALL MUST RETAIN THE LOADS FROM 3 SPANS

$$EQ = \frac{965 \text{ (3 SPANS)}}{37' \text{ ABUT}} \times 0.29 = 22.7 \text{ KLF}$$

$\uparrow$ DL $\qquad$ $\uparrow$ As

$A_{c_1} = 750\text{mm} \times 1' = 29.5" \times 12" = 354 \text{ IN}^2$

$C = 0.28\text{KSI}$
$M = 1.0$
$K_1 = 0.3$
$K_2 = 1.8$

AASHTO 5.8.4.3, BASED ON CONSTRUCTION JOINT AT BASE OF BACKWALL

$AV = 0.31 \text{ IN}^2/\text{FT} \times 2 = 0.62 \text{ IN}^2/\text{FT}$
$P_c = 0$

$\phi V_n = 1.0[0.28 \times 354 + 1.0 \times 0.62 \times 60] = 99.1 + 37.2 = 136.3^K$ OK

$$a = \frac{0.31(60)}{.85(4)(60)} = 0.456"$$

$d = 750 - 50 - 8 = 692\text{mm} = 27.2"$

$\phi M_n = .9(0.31)(60)(27.2 - 0.456/2)/12 = 37.6 \text{ K}\cdot\text{FT}$

$M_U = 22.7 \times 2.6' = 59.02 \text{ K}\cdot\text{FT}$ ∴ NO GOOD (64% OF CAPACITY)

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Framingham, MA 01701-9005
(508) 903-2000

JOB: AESP BRIDGE 9
SHEET NO. ____ OF ____
CALCULATED BY: APL DATE: 9/29/14
CHECKED BY: SAM DATE: 9/30/14
SCALE: ____

SEISMIC RESTRAINT DESIGN – BRIDGE 9 (CONT'D)

– LONGITUDINAL

TRY #5@6" FLEXURAL REINFORCEMENT

$$a = \frac{0.62(60)}{.85(4)(60)} = \underline{0.91"} \qquad d = 27.2"$$

$$\phi M_N = .9(.62)(60)(27.2 - 0.91/2)/12 = 74.6\ K{\cdot}FT$$

$$M_U = 59.02\ K{\cdot}FT < \phi M_N \quad \underline{\underline{OK}}$$

$\boxed{\text{RECOMMENDATION}}$

FOR BRIDGE 9, USE THE SAME DETAILS FOR SEISMIC RESTRAINT AS BRIDGE 10, EXCEPT INCREASE THE BACKWALL FLEXURAL REINFORCEMENT TO 16φ@150

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB AESP BRIDGE 9
SHEET NO. ______ OF ______
CALCULATED BY APL DATE 9/29/14
CHECKED BY SAM DATE 9/30/14
SCALE ______

SUBSTRUCTURE DESIGN

ABUTMENT DESIGN

* SUPERSTRUCTURE DEAD LOADS ON ABUTMENT
* SUPERSTRUCTURE LIVE LOADS ON ABUTMENT
* ABUTMENT STABILITY DESIGN

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/10/2014

References: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Notes: This spreadsheet computes the loads on an abutment, considering the spans left or right of the abutment is simply supported.

SUPERSTRUCTURE DEAD LOAD

Abutment Length = 36.90 ft

Length =	50400	mm	165.31	ft		
Spans =	2		2			
Span Length =	16800	mm	55.10	ft		
No of Beams =	6		6			
b =	600	mm	1.97	ft	23.62	in
d =	1500	mm	4.92	ft	59.04	in
Spacing =	1850	mm	6.07	ft	72.82	in
Distance from Ext Beam to Ext Beam =	9250	mm	30.34	ft	364.08	in
Overhang, OH =	850	mm	2.79	ft	33.46	in
Clear Overhang, OH_cl =	550	mm	1.80	ft	21.65	in
Clear Spacing bw Beams =	1250	mm	4.10	ft	49.20	
Deck Thickness, ts =	225	mm	0.74	ft	8.86	in
Barrier Height =	1400	mm	4.59	ft	55.10	in
Sidewalk Height =	275	mm	0.90	ft	10.82	in

Width =	11250	mm	36.90	ft
Barrier =	225	mm	0.74	ft
Sidewalk =	1200	mm	3.94	ft
Barrier + Sidewalk =	1425	mm	4.67	ft
Roadway =	8100	mm	26.57	ft
No of Lanes	2	mm	0.01	ft
Lane Width =	3657.6	mm	12.00	ft
Shoulder =	392.4	mm	1.29	ft

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BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/10/2014

SUPERSTRUCTURE DEAD LOAD

CAT		Width ft	Height ft	Length ft	Volume cf	Unit Weight lbs/cf	Weight Kips	Qty #	Total Kips	DC Kips	DW Kips
DC	Beams / Girders	1.97	4.92	55.10	533.55	150	80.03	6	480.19	480.19	
DC	Sidewalks	3.94	0.90	55.10	195.40	150	29.31	2	58.62	58.62	
DC	Safety Curbs	0.00	0.00	0.00	0.00	150	0.00	0	0.00	0.00	
DC	Barriers	0.90	4.59	55.10	228.14	150	34.22	2	68.44	68.44	
DW	Wearing Surface	26.57	0.18	55.10	263.54	165	43.48	1	43.48		43.48
DC	End Diaphragms	4.10	4.92	2.46	49.62	150	7.44	10	74.43	74.43	
DC	Intermediate Diaphragms	4.10	4.92	0.98	19.77	150	2.97	5	14.83	14.83	
DW	Utilities				0.00	0	0.00	0	0.00		0.00
DW	Stay-In-Place Forms	0.00	0.00	0.00	0.00	0	0.00	0	0.00		0.00
DC	Deck	36.90	0.74	55.10	1500.60	150	225.09	1	225.09	225.09	
					0.00		0.00		0.00		
					0.00		0.00		0.00		
					0.00		0.00		0.00		
									965.09	921.61	43.48
										965.09	

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/10/2014

SUPERSTRUCTURE DEAD LOAD

		DC (kips)	DW (Kips)	DL (kips)	
DC	Beams / Girders	480.19			
DC	Sidewalks	58.62			
DC	Safety Curbs	0.00	0.00		
DC	Barriers	68.44			
DW	Wearing Surface		43.48		
DC	End Diaphragms	74.43			
DC	Intermediate Diaphragms	14.83			
DW	Utilities		0.00		
DW	Stay-In-Place Forms		0.00		
DC	Deck	225.09			
		DC	DW	DL	
Total Superstructure Dead Load		921.61	43.48	965.09	kips
Total Dead Load / Abutment		460.80	21.74	482.55	kips / Abutment
Length of Abutment		36.90	36.90	36.90	Abutment Length
Total (DL / Linear Foot of Abutment)		12.49	0.59	13.08	
Total (DL / Linear Foot of Pier)		24.98	1.18	26.15	
Abutment Design					
Design For: Abutment Loading		**12.49**	**0.59**	**13.08**	kips/LF of Abutment

<-- Superstructure Dead Load (Input Tab)

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BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/3/2014

References: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Notes: This spreadsheet computes the loads on an abutment, considering the spans left or right of the abutment is simply supported.

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Live Load, LL

Type of Truck: HL	93	
Roadway Width =	26.24	ft
Design Lane Width =	12	ft
Roadway / Lane Width =	2.19	
Use --> No of Lanes =	2	
Multiple Presence Factor, m =	1	

Table 3.6.1.1.2-1—Multiple Presence Factors, *m*

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65

Section 3.6.1.2.2

Section 3.6.2.1

Section 3.6.1.1.2

Truck Loading:

	Left/Right Span		
Span Length, L =	55.10	ft	
Dynamic Load Allowance, (IM) =	1.33		
Number of Lanes =	2		
Multiple Presence Factor, m =	1.00		
Vmax =	59.1	kips / Lane	<-- T3.3.1.2 Shear & End Reactions
Vmax =	118.20	kips	<- Vmax * m * # of lanes
Reaction, LL V =	118.20	kips	
Reaction, (LL+IM) V =	157.2	kips	<-- IM * V
Total Reaction, Truck (LL) =	118.2	kips	
Total Reaction, Truck (LL+IM) =	157.2	kips	

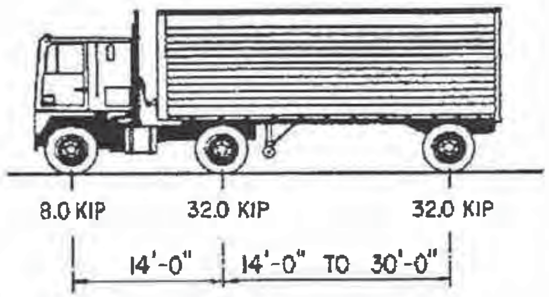


Figure 3.6.1.2.2-1

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/3/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Tandem Loading: Section 3.6.1.2.3

	Left/Right Span			
L =	55.10	ft		
Dynamic Load Allowance, (IM) =	1.33			Section 3.6.2.1
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
P1 =	25	kips		
P2 =	25	kips		
Axle Spacing =	4	ft		
Vmax =	48.19	kips/ Lane		
Vmax =	96.37	Kips	<- Vmax * m * # of lanes	
Reaction, LL V =	96.37	kips		
Reaction, (LL+IM) V =	128.17	kips	<-- IM * V	
Total Reaction, Tandem (LL) =	96.4	kips		
Total Reaction, Tandem (LL+IM) =	128.2	kips		

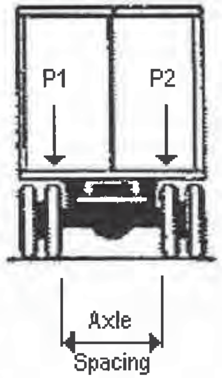
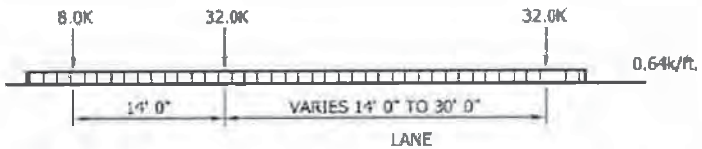


Figure 3.6.1.2.2-1

Live Load, LL (cont.)

Lane Loading: Section 3.6.1.2.4

	Left/Right Span			
L =	55.10	ft		
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
Lane Load =	0.64	klf		
Vmax =	17.63	kips/ Lane		
Vmax =	35.27	Kips	<- Vmax * m * # of lanes	
Reaction, Lane Load (LL) =	35.3	kips		
Total Reaction, Lane Load (LL) =	35.3	kips		



BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/3/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Pedestrian Live Load

Pedestrian Live Load, PL = 0.075 ksf <--- per AASHTO 3.6.1.6 for Sidewalks with a Width >= 2.0 ft

Width of Sidewalk = 0.00 ft

PL = 0.000 klf

Length of Sidewalk = 55.10 ft

Bridge Width = 36.90 ft

PL = 0.00 kips --> PL / Abutment = 0.00 kips --> PL / LF of Abutment = 0.00 klf

Live Loads

	LL	IM	LL + IM
Truck	59.10	1.33	78.60
Tandem	48.19	1.33	64.09
Lane	17.63	1	17.63
Truck + Lane	76.73		96.24
Tandem + lane	65.82		81.72
Max	76.73		96.24

Max = 96.24 kips

No of Lanes = 2.00

m = 1.00

LL+I = 192.47 kips

Abutment Length = 36.90 ft

LL+ I = 5.22 klf

LL + I + PL = 5.22 klf **<-- INPUT LOAD**

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/3/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - LATERAL FORCES

Braking Force, BR — Section 3.6.4

Notes: Dynamic Load Allowance increase not required. AASHTO3.6.2.1
Braking Force ONLY applies to fixed bearings
Braking Force includes multiple presence factor

25% Axle Weight of Design Truck =	25%	18.00	kips	Design Truck Axle Weight =	72
25% Axle Weight of Design Tandem =	25%	12.50	kips	Design Tandem Axle Weight =	50
5% (Axle Weight of Design Truck + Lane Load) =	5%	5.36	kips	Design Truck + Lane Axle Weight =	107.27
5% (Axle Weight of Design Tandem Load + Lane Load) =	5%	4.26	kips	Design Tandem + Lane Axle Weight =	85.27

Braking Force on Abutment (BR) =	18.00	kips	<---- 25% Axle Weight of Design Truck	
Number of Lanes =	2			
Multiple Presence Factor, m =	1			
Breaking Force Applied to 2 Abutements =	Y			
BR =	**0.49**	**klf**	<--- BR / Abutment Length	**<-- Input Load**

Location of Load Application = *0.00* *ft above Bridge Seat*

SUPERSTRUCTURE LOADING ON ABUTMENT - LATERAL FORCES - EQ

EQ = 2.15 klf **<---See Hand Caclulations**

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS— SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	485.3(b)	56.0(b)
2	16.0(b)	32.0(b)	44	520.9(b)	56.7(b)
3	24.0(b)	32.0(b)	46	556.5(b)	57.3(b)
4	32.0(b)	32.0(b)	48	592.1(b)	58.0(b)
5	40.0(b)	32.0(b)	50	627.9(b)	58.5(b)
6	48.0(b)	32.0(b)	52	663.6(b)	59.1(b)
7	56.0(b)	32.0(b)	54	699.3(b)	59.6(b)
8	64.0(b)	32.0(b)	56	735.1(b)	60.0(b)
9	72.0(b)	32.0(b)	58	770.8(b)	60.4(b)
10	80.0(b)	32.0(b)	60	806.5(b)	60.8(b)
11	88.0(b)	32.0(b)	62	842.4(b)	61.2(b)
12	96.0(b)	32.0(b)	64	878.1(b)	61.5(b)
13	104.0(b)	32.0(b)	66	914.0(b)	61.9(b)
14	112.0(b)	32.0(b)	68	949.7(b)	62.1(b)
15	120.0(b)	34.1(b)	70	985.6(b)	62.4(b)
16	128.0(b)	36.0(b)	75	1,075.1(b)	63.1(b)
17	136.0(b)	37.7(b)	80	1,164.9(b)	63.6(b)
18	144.0(b)	39.1(b)	85	1,254.7(b)	64.1(b)
19	152.0(b)	40.4(b)	90	1,344.4(b)	64.5(b)
20	160.0(b)	41.6(b)	95	1,434.1(b)	64.9(b)
21	168.0(b)	42.7(b)	100	1,524.0(b)	65.3(b)
22	176.0(b)	43.6(b)	110	1,703.6(b)	65.9(b)
23	184.0(b)	44.5(b)	120	1,883.3(b)	66.4(b)
24	192.7(b)	45.3(b)	130	2,063.1(b)	67.6
25	207.4(b)	46.1(b)	140	2,242.8(b)	70.8
26	222.2(b)	46.8(b)	150	2,475.1	74.0
27	237.0(b)	47.4(b)	160	2,768.0	77.2
28	252.0(b)	48.0(b)	170	3,077.1	80.4
29	267.0(b)	48.8(b)	180	3,402.1	83.6
30	282.1(b)	49.6(b)	190	3,743.1	86.8
31	297.3(b)	50.3(b)	200	4,100.0	90.0
32	312.5(b)	51.0(b)	220	4,862.0	96.4
33	327.8(b)	51.6(b)	240	5,688.0	102.8
34	343.5(b)	52.2(b)	260	6,578.0	109.2
35	361.2(b)	52.8(b)	280	7,532.0	115.6
36	378.9(b)	53.3(b)	300	8,550.0	122.0
37	396.6(b)	53.8(b)			
38	414.3(b)	54.3(b)			
39	432.1(b)	54.8(b)			
40	449.8(b)	55.2(b)			

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BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB
PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)
GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

Table 3.3.1.1
Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions
Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
1	8.0	6.3	0.1	8.1	0.50	32.0	25.0	0.3	32.3
2	16.0	12.5	0.3	16.3	0.50	32.0	25.0	0.6	32.6
3	24.0	18.8	0.7	24.7	0.50	32.0	25.0	1.0	33.0
4	32.0	25.0	1.3	33.3	0.50	32.0	25.0	1.3	33.3
5	40.0	31.3	2.0	42.0	0.50	32.0	30.0	1.6	33.6
6	48.0	37.5	2.9	50.9	0.50	32.0	33.3	1.9	35.3
7	56.0	43.8	3.9	59.9	0.50	32.0	35.7	2.2	38.0
8	64.0	50.0	5.1	69.1	0.50	32.0	37.5	2.6	40.1
9	72.0	62.5	6.5	78.5	0.50	32.0	38.9	2.9	41.8
10	80.0	75.0	8.0	88.0	0.50	32.0	40.0	3.2	43.2
11	84.5	92.0	9.3	101.3	0.40	32.0	40.9	3.5	44.4
12	92.2	104.0	11.1	115.1	0.40	32.0	41.7	3.8	45.5
13	103.0	115.9	13.4	129.3	0.45	32.0	42.3	4.2	46.5
14	110.9	128.3	15.5	143.8	0.45	32.0	42.9	4.5	47.3
15	118.8	140.6	17.8	158.4	0.45	34.1	43.3	4.8	48.1
16	126.7	153.0	20.3	173.3	0.45	36.0	43.8	5.1	48.9
17	134.6	165.4	22.9	188.3	0.45	37.6	44.1	5.4	49.6
18	142.6	177.8	25.7	203.4	0.45	39.1	44.4	5.8	50.2
19	150.5	190.1	28.6	218.7	0.45	40.4	44.7	6.1	50.8
20	158.4	202.5	31.7	234.2	0.45	41.6	45.0	6.4	51.4
21	166.3	214.9	34.9	249.8	0.45	42.7	45.2	6.7	52.0
22	174.2	227.3	38.3	265.6	0.45	43.6	45.5	7.0	52.5
23	182.2	239.6	41.9	281.5	0.45	44.5	45.7	7.4	53.0
24	190.1	252.0	45.6	297.6	0.45	45.3	45.8	7.7	53.5
25	198.0	264.4	49.5	313.9	0.45	46.1	46.0	8.0	54.1
26	210.2	276.8	53.5	330.3	0.45	46.8	46.2	8.3	55.1
27	226.1	289.1	57.7	346.9	0.45	47.4	46.3	8.6	56.0
28	241.9	301.5	62.1	363.6	0.45	48.0	46.4	9.0	57.0
29	257.8	313.9	66.6	380.5	0.45	48.8	46.6	9.3	58.1
30	273.6	326.3	71.3	397.5	0.45	49.6	46.7	9.6	59.2
31	289.4	338.6	76.1	414.7	0.45	50.3	46.8	9.9	60.2
32	307.0	351.0	81.1	432.1	0.45	51.0	46.9	10.2	61.2
33	324.9	363.4	86.2	449.6	0.45	51.6	47.0	10.6	62.2
34	332.0	375.0	92.5	467.5	0.50	52.2	47.1	10.9	63.1
35	350.0	387.5	98.0	485.5	0.50	52.8	47.1	11.2	64.0
36	368.0	400.0	103.7	503.7	0.50	53.3	47.2	11.5	64.9
37	386.0	412.5	109.5	522.0	0.50	53.8	47.3	11.8	65.7
38	404.0	425.0	115.5	540.5	0.50	54.3	47.4	12.2	66.5
39	422.0	437.5	121.7	559.2	0.50	54.8	47.4	12.5	67.2
40	440.0	450.0	128.0	578.0	0.50	55.2	47.5	12.8	68.0

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

Table 3.3.1.2

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions

Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
42	485.2	474.6	139.7	624.9	0.45	56.0	47.6	13.4	69.4
44	520.9	499.5	153.3	674.2	0.45	56.7	47.7	14.1	70.8
46	556.5	524.3	167.6	724.1	0.45	57.4	47.8	14.7	72.1
48	592.2	549.0	182.5	774.8	0.45	58.0	47.9	15.4	73.4
50	627.8	573.8	198.0	825.8	0.45	58.6	48.0	16.0	74.6
52	663.4	598.5	214.2	877.6	0.45	59.1	48.1	16.6	75.7
54	699.1	623.3	230.9	930.0	0.45	59.6	48.1	17.3	76.8
56	734.7	648.0	248.4	983.1	0.45	60.0	48.2	17.9	77.9
58	770.4	672.8	266.4	1036.8	0.45	60.4	48.3	18.6	79.0
60	806.0	697.5	285.1	1091.1	0.45	60.8	48.3	19.2	80.0
62	841.6	722.3	304.4	1146.1	0.45	61.2	48.4	19.8	81.0
64	877.3	747.0	324.4	1201.7	0.45	61.5	48.4	20.5	82.0
66	912.9	771.8	345.0	1257.9	0.45	61.8	48.5	21.1	82.9
68	948.6	796.5	366.2	1314.8	0.45	62.1	48.5	21.8	83.9
70	984.2	821.3	388.1	1372.3	0.45	62.4	48.6	22.4	84.8
75	1070.0	887.5	450.0	1520.0	0.50	63.0	48.7	24.0	87.0
80	1160.0	950.0	512.0	1672.0	0.50	63.6	48.8	25.6	89.2
85	1250.0	1012.5	578.0	1828.0	0.50	64.1	48.8	27.2	91.3
90	1340.0	1075.0	648.0	1988.0	0.50	64.5	48.9	28.8	93.3
95	1430.0	1137.5	722.0	2152.0	0.50	64.9	48.9	30.4	95.3
100	1520.0	1200.0	800.0	2320.0	0.50	65.3	49.0	32.0	97.3
110	1700.0	1325.0	968.0	2668.0	0.50	65.9	49.1	35.2	101.1
120	1880.0	1450.0	1152.0	3032.0	0.50	66.4	49.2	38.4	104.8
130	2060.0	1575.0	1352.0	3412.0	0.50	66.8	49.2	41.6	108.4
140	2240.0	1700.0	1568.0	3808.0	0.50	67.2	49.3	44.8	112.0
150	2420.0	1825.0	1800.0	4220.0	0.50	67.5	49.3	48.0	115.5
160	2600.0	1950.0	2048.0	4648.0	0.50	67.8	49.4	51.2	119.0
170	2780.0	2075.0	2312.0	5092.0	0.50	68.0	49.4	54.4	122.4
180	2960.0	2200.0	2592.0	5552.0	0.50	68.3	49.4	57.6	125.9
190	3140.0	2325.0	2888.0	6028.0	0.50	68.5	49.5	60.8	129.3
200	3320.0	2450.0	3200.0	6520.0	0.50	68.6	49.5	64.0	132.6

http://www.dot.nd.gov/manuals/bridge/lrfd-bridge-design/Section03A.pdf

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions

General Notes: **This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).**

Notes for Khost Bridge No 10

Project Notes: For the Design: Design was based on the West Abutment

General Design Parameters

Input Section : 1.0

GEOMETRY INFORMATION INPUT:

PROPOSED TOP OF ROADWAY ELEV:		5975.61 ft	1821.83 m
PROPOSED TOP OF BACKWALL ELEV:		5974.67	1821.55 m
PROPOSED BRIDGE SEAT ELEV:	H_Backwall = 5.14 ft	5969.53 ft	1819.98 m
PROPOSED TOP OF FOOTING ELEV:	H_Footing = 4.92 ft	5949.92 ft	1814.00 m
PROPOSED BOT. OF FOOTING ELEV:		5945.00 ft	1812.50 m
ELEVATION OF HIGH WATER:	FOR NO WATER = 0.00	5966.88	1819.17 m
PROPOSED BRIDGE SEAT WIDTH:		1.68 ft	0.51 m
PROPOSED BACKWALL WIDTH:		2.46 ft	0.75 m
ABUTMENT/WALL DESIGN LENGTH:	1.00 Ft Actual Length=	36.90 ft	11.25 m
FOOTING LENGTH:		44.12 ft	13.45 m

DW CALCULATION INPUT:

WEARING SURFACE DEPTH:	2.13 IN x 1. Layers	0.18 ft <---	0.05 m
ROADWAY WIDTH:		26.24 ft	8.00 m
BRIDGE SPAN:	**Total Length = 165.312** **<-- 3 Spans @**	55.10 ft	16.80 m
NUMBER OF GIRDERS:		6	

MATERIAL PROPERTIES:

CUBIC WEIGHT CONCRETE:	150.00 pcf
COMP. STRENGTH OF CONC. = F'c:	4.00 ksi
MAXIMUM SIZE OF COARSE AGGREGATE	1.50 in
TENSILE STRENGTH OF REBAR = Fy:	60.00 ksi
CUBIC WEIGHT OF HOT MIX ASPHALT (HMA):	165.00 pcf

GEOTECHNICAL INFORMATION:

NOMINAL BEARING RESISTANCE (CAPACITY),qn:	17.78 ksf	<-- Per Geotech Report
FACTORED BEARING RESISTANCE, qr :	8.00 ksf	<-- Per Geotech Report
WEIGHT OF SOIL BACKFILL:	130.00 Lbs/CF	<-- Per Geotech Report
WALL ON ROCK?	N (Y OR N)	
WALL ON PILES?	N (Y OR N)	
GRAVITY WALL?	N (Y OR N)	
BETA: SLOPE OF BACKFILL:	0.00 DEG	<-- Per Civil
THETA: BATTER ANGLE BACKWALL:	78.01 DEG	AASHTO Table 3.11.5.3-1
PHI: FRICTION ANGLE OF BACKFILL:	33.00 DEG	<-- Per Geotech Report
DELTA: ANGLE BACKWALL FRICTION:	22.00 DEG	<-- Assumed δ=2/3 (ϕ)

Fill-in for Abutment / Pier Design

Khost Bridge No.10

CANTILEVER ABUTMENT DESIGN	Y
GRAVITY ABUTMENT DESIGN	N
CANTILEVER WALL DESIGN	N
GRAVITY WALL DESIGN	N
PIER DESIGN	N

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

General Loading Parameters

Input Section : 2.0

LIVE LOAD INFORMATION:

APPROACH SLAB:	Y	(Y OR N)
ROADWAY WITHIN H/2 OF TOP OF WALL:	Y	(Y OR N)
Live Load Surcharge to be Considered?:	Y	
SURCHARGE HEIGHT:	2.00	ft REF: Table 3.11.6.4-1
Construction Surcharge, q:	250.00	psf REF: C3.4.2.1

SEISMIC LOAD INFORMATION:

WALL RESTRAINED HORZ. MOVMT.(Y/N):	N	(Y OR N)
SEISMIC ACCELERATION COEFF. A:	0.290	REF: FIG.3.10.2.1-2, AASHTO
SEISMIC CATEGORY:	D	<---**Assumed based on Location & AASHTO Seimic Design Guide**

RAILING CLASS: S3-TL4 (CT) (PER MASSDOT LRFD BRIDGE MANUAL PART 1) 3.3.2.2 **<--- N/A**

Horizontal Railing Design Load	0.00	kips
Horizontal Railing Impact Length	0.00	ft
Wall Height+Rail Height	0.00	ft
Distributed Horizontal Railing Design Load @ top of wall	0.00	klf
Distributed Horizontal Railing Design Load @ bottom of wall	0.00	klf/wall height
Railing Dead Load	0.00	
Additional Moment From Railing Impact	0.00	<--- **Note:** The added moment from top of railing to bottom of railing is distributed along bottom of footing*

SURCHARGE HEIGHT (Per ASSHTO 3.11.6.4 Live Load Surchage)

ABUTMENTS ---> Table 3.11.6.4-1

Table 3.11.6.4-1 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpindicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5	4
10	3
>20	2

Surcharge Height = 2.00 ft

RETAINING WALLS - --> Table 3.11.6.4-2

See Table 3.11.6.4-2 for Equivalent Height of Soil for Vechicular Loading on Retaining Walls Parallel to Traffic.

Retaining Wall Height (ft)	heq (ft) Distance from wall backface to edge of traffic	
	0.0 ft	≥1.0 ft
5	5	2
10	3.5	2
>20	2	2

Distance from wall backface to edge of traffic = 0.0 ft

Surcharge Height = 2.00 ft

Note: See 3.11.6.5 for Possible Reduction of Surcharge

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Superstructure Loading Parameters

Input Section : 3.0

ADDITIONAL LOADS ON STRUCTURE

(load is per linear foot of structure (Abutment/ Pier/ Wall) NOT the Footing, arm from front edge of bridge seat)

LOADS		LOAD (klf)	ARM (feet)			
(DC+DW), SUPERSTRUCT. DEAD LOAD:	DL	13.08	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DC (Structural Components & nonstructural attachments)	DC	12.49	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DW (Wearing Surface & Utilities)	DW	0.59	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
(LL+IM+PL), LIVE LOAD, IMPACT AND PED LL:	LL+IM+PL	5.22	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
WS, WIND LOAD ON STRUCTURE:	WS	0.40	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
WL, WIND LOAD ON LIVE LOAD:	WL	0.08	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
BR, BREAKING LOAD :	BR	0.49	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
TU, THERMAL FORCE:	TU	0.37	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
EQ, SEISMIC LOAD ON SUPERSTRUCTURE:	EQ	2.15	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
CT, VEHICLE COLLISION LOAD	CT	0.00	0.00	Distance above top pf wall equal to the height of rail	Include =	Y

Note: Per AASHTO 11.5.1, abutments and retaining walls should be designed for EH, WA, LS, DS, DC, TU, EQ. Therefore, including wind and breaking forces is conservative. Say OK

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

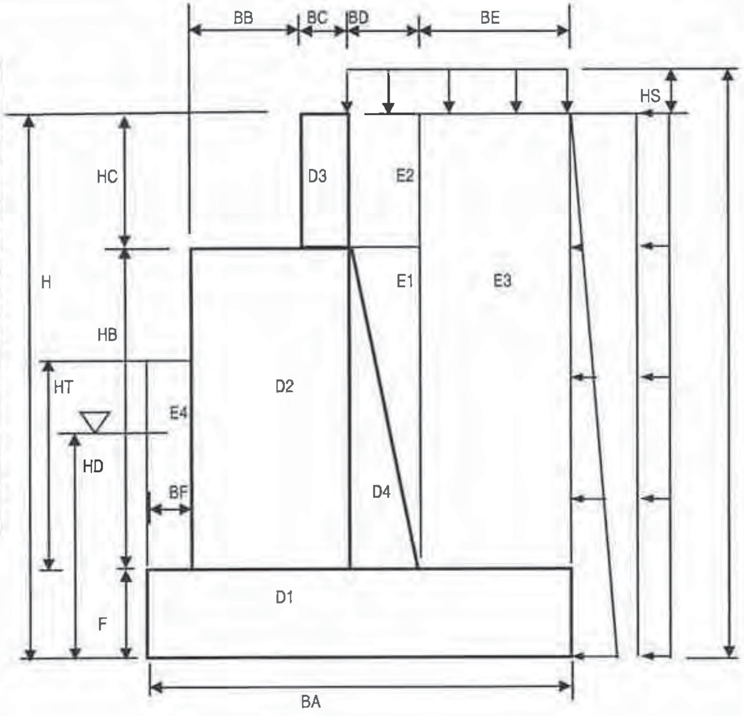
Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Abutment Geometry

Input Section : 4.0

CALCULATION OF WALL AND BACKFILL GEOMETRY:

		Prelim Size	User Adjust	Final Size (ft)	Approx Size (mm)
HEIGHT OF ABUTMENT / WALL, H:	H =	29.668	0.00	29.67	9100
HEIGHT OF FOOTING, F:	F =	4.920	0.00	4.92	1500
HEIGHT OF STEM, HB:	HB =	19.608	0.00	19.61	5980
HEIGHT OF BACKWALL, HC:	HC =	5.140	0.00	5.14	1570
HEIGHT OF HIGH WATER, HD:	HD =	21.878	0.00	21.88	6670
HEIGHT OF SURCHARGE, HS:	HS =	2.000	0.00	2.00	610
WIDTH OF FOOTING, BA:	BA =	22.950	0.00	22.95	7000
WIDTH OF BRIDGE SEAT, BB:	BB =	1.679	0.00	1.68	520
WIDTH OF BACKWALL, BC:	BC =	2.460	0.00	2.46	750
WIDTH OF BATTER OF STEM, BD:	BD =	5.045	0.00	5.05	1540
WIDTH OF FOOTING HEEL, BE:	BE =	9.180	0.00	9.18	2800
WIDTH OF FOOTING TOE, BF:	BF =	3.936	0.00	3.94	1200
HEIGHT OF SOIL OVER TOE, HT:	HT =	4.953	0.00	4.95	1510
HEIGHT OF SOIL OVER HEEL, HH:	HH =	24.748	0.00	24.75	7550
HEIGHT OF SOIL AT BACKFACE FACE (HEEL), HS1	Hss1 =			29.668	9100
HEIGHT OF SOIL AT FRONT FACE FACE (TOE), HS2	Hss2 =			9.873	3100



OVERALL QUANTITIES:

WEIGHT OF CONCRETE WALL/L.F.:	38.427	Kips per l.f.
CONCRETE QUANTITY / L.F.:	9.488	C.Y. per l.f.

SUMMARY OF QUANTITIES:

STEEL / L.F. =	1210.255	LBS/L.F.
CONC. / L.F =	9.488	C.Y./L.F.

Geometry Check: Check Width: **NG**

Check Height: **ok**

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Notes for Khost Bridge No 10

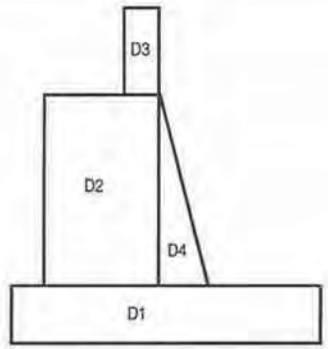
Calculate Dead Loads

Primary Loads Section : 1.0

Superstructure Loads:

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	Superstructure	12.49	5.58	69.63			
DW	Superstructure	0.59	5.58	3.29			

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.



Substructure Loads:

AREA #		Vertical: Volume (CF)	γ_{conc} (pcf)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	D1	112.91	150.00	16.94	11.48	194.35			
	D2	81.16	150.00	12.17	6.01	73.11			
	D3	12.64	150.00	1.90	6.85	12.98			
	D4	49.46	150.00	7.42	9.76	72.39			
	Subtotal Concrete			**38.43**		**352.83**			

Total Dead Load:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	**50.91**		**422.46**			
TOTAL DW (Super)	**0.59**		**3.29**			
TOTAL DC (Substr. Only - Construction)	**38.43**		**352.83**			

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Earth Loads

Primary Loads Section : 2.0

Compute Horizontal Earth Pressure, EH:

Coulomb's Active Earth Pressure: (per MHD 3.1.5 and AASHTO 3.11.5.3)

PHI, $\phi f'$ =	33.00	Degrees,	Rad =	0.58
DELTA, δ =	22.00	Degrees,	Rad =	0.38
BETA, β =	0.00	Degrees,	Rad =	0.00
THETA, θ =	78.01	Degrees,	Rad =	1.36
Γ (per AASHTO Eq. 3.11.5.3-2)=	3.03			
Ka (per AASHTO Eq. 3.11.5.3-1)=	**0.362**			

Earth Pressure Coefficient to be Used for Design per MassDOT

All Walls on Rock	ko	0.455	
All Walls on Piles	ko	0.455	
Cantilever Walls < than 16' in Height	0.5*(Ko + Ka)	0.409	
Cantilever Walls > than 16' in Height	Ka	0.362	<-- USE
Gravity wall supported on Spread Footing	Ka	0.362	

At-Rest Earth Pressure Coeff:

Ko = **0.455**

Earth Pressure Coefficient to be Used for Design:

WALL ON LEDGE:	N	(Y OR N)
WALL ON PILES:	N	(Y OR N)
Wall Height:	29.668	ft
Earth pressure Type:	Ka	
Ke =	**0.362**	<===== **Governs**

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Ko =	0.49	
Ka =	0.32	
Ke (geotech) =	**0.320**	<=== **Does not govern.**

Compute Lateral Earth Pressure:

Application of lateral earth pressure shall be per AASHTO Figure C3.11.5.3-1. This shows a different application for Gravity and Cantilever (semi-gravity) walls.
Note that the reduction in lateral earth pressures due to the water table is not included in this section. It is included in the WA (Bouyancy) section of this design.

Cantilever (semi-gravity) Walls:

THIS SECTION IS FOR CANTILEVER OR SEMI-GRAVITY WALLS ONLY

Load inclination from horizontal, min = ϕ/3 =	11.00	degrees
Load inclination from horizontal, max = ϕ*2/3 =	22.00	degrees
GAMMA =	130.00	pcf
H = Soil Height at Back face, Hss1	**29.67**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	**20.72**	kips
Arm for Horiz Load above BOF = H/3 =	9.89	ft
Arm for Vert Load from Toe = BA =	22.95	ft

Consider minimum inclination for Sliding, Overturning and Bearing Pressure:

Vertical Component, Pav = Pa*sin(ϕ/3) =	3.95	klf
Horizontal Component, Pah = Pa*cos(ϕ/3) =	20.34	klf

Consider maximum inclination for Footing Heel Reinforcement:

Vertical Component, Pav = Pa*sin(ϕ*2/3) =	7.76	klf
Horizontal Component, Pah = Pa*cos(ϕ*2/3) =	19.21	klf

Gravity Walls:

N/A ---> THIS SECTION IS FOR GRAVITY WALLS ONLY

Load inclination from horizontal = δ + (90–θ) =	33.99	degrees
GAMMA =	130.00	pcf
H =	**29.67**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	**20.72**	kips
Arm for Horiz Load above BOF = H/3 =	**9.89**	ft
Arm for Vert Load from Toe=(BF+BB+BC+BD*2/3) =	11.44	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Pav = Pa*sin(δ+(90–θ)) =	11.58	klf
Horizontal Component, Pah = Pa*cos(δ+(90–θ)) =	17.18	klf
Is the wall a Gravity Wall?	N	

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.1

Include Passive Earth Pressure	Y
Pp Factor	1

ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kp = Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
H = Hss2= Height of Soil at Front Face - 1'	8.87	ft

Lateral EQ Load, Pp = 1/2*γ*Kp*H^2= 16.02 klf < Pah ----> Use Pp as calculated ---> **Pp = 16.02 klf**

Arm for Horiz Load above BOF = H/3 = 2.96 ft (AASHTO pg 11-112)

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.2

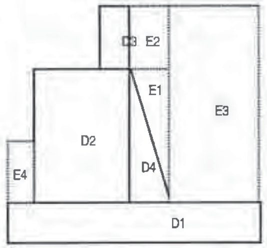
Horizontal Earth Pressure, EH:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EH: Pa	3.95	22.95	90.73	20.34	9.89	201.14
EH: Pp			0.00	-16.02	2.96	-47.38
EH (For all cases except heel reinforcement):	3.95	**22.95**	90.73	4.32	12.85	153.75
EH: Pa	7.76	22.95	178.13	19.21	9.89	189.98
EH: Pp			0.00			0.00
EH (For Heel Reinforcement):	7.76	**22.95**	178.13	**19.21**	**9.89**	189.98

<=== Note, Based on AASHTO Figure C11.5.6-1, both the vertical and horizontal components of EH should be included here because they carry the same load factor.

Vertical Earth Pressure, EV:

AREA #		Volume (CF)	γ_{SOIL} (plf)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EV	E1	49.46	130.00	6.43	11.44	73.55			
	E2	25.93	130.00	3.37	10.60	35.72			
	E3	227.19	130.00	29.53	17.71	523.05			
	E4	19.50	130.00	2.53	1.97	4.99			
TOTAL EV				**41.87**		**637.31**			



Note, **per AASHTO 11.6.1.2,** the weight of the soil over the battered portion of the stem or over the base of a footing may be considered as part of the effective weight of the abutment. This is consistant with design.

Earth Surcharge, ES: (This applies for construction case only)

q =	250.00	psf
Uniform Load on Wall, p=Ke*q =	0.091	ksf
Wall Height, H =	29.67	Feet
Heel Length, BE =	9.18	Feet
Footing Width, BA =	22.95	Feet
Wall Length Considered =	1.00	ft

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
ES	$P_{con}(h) = p*H*Length =$				2.69	14.83	39.84
	$P_{con}(v) = q*BE*Length =$	3.56	15.84	56.32			
TOTAL ES		**3.56**		**56.32**	**2.69**		**39.84**

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Live Loads

Primary Loads Section : 3.0

Superstructure Loads:

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LL+IM+PL	Superstructure	5.22	5.58	29.08			
BR	Superstructure				0.488	24.53	11.96

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Live Load Surcharge Loads: LS

Per AASHTO 3.11.6.4, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall.
If the surcharge is for highway, the intensity of the load shall be consistent with provisions of Article 3.6.1.2. See Tables 3.11.6.4-1 and 3.11.6.4-2 for equivalent heights.

Compute Horizontal Live Load Surcharge: (To be used for bearing pressure and sliding load cases):

Ke =	0.362	
Unit Weight of Soil, γ =	130.000	pcf
Surcharge Height, heq =	2.00	Feet
LS(h) =(Ke)(γ)(heq)*H =	2.79	kips
Moment arm = H/2 =	14.83	kips

Compute Vertical Live Load Surcharge: (To be used for bearing pressure cases only):

LS(v) =(γ)(heq)(BD+BE) =	3.70	kips
Moment arm = Ba-(BD+BE)/2 =	15.84	kips

Compute Vertical Live Load Surcharge: (To be used for heel reinf cases only):

LS(v) =(γ)(heq)(BE) =	2.39	kips
Moment arm (to back of batter) = BE/2 =	4.59	kips

Live Load Surcharge, LS: Summary

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LS	LS(v)	3.70	15.84	58.57			
	LS(h)				2.79	14.83	41.44

Total Live Load Load:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL LL+IM+PED+BR+LS	8.91		87.66	3.28		53.40
TOTAL LL+IM+PED+BR+LS (Sliding Only)	5.22		29.08	3.28		53.40
TOTAL LS (Heel Reinf Only)	2.39	4.59	10.96			

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Buoyancy Forces

Primary Loads Section : 4.0

HEIGHT OF STEM AT HIGH WATER:	16.96	
HEIGHT OF FOOTING AT HIGH WATER:	4.92	
WIDTH OF FOOTING, BA	22.95	
SOIL WEIGHT - WATER WEIGHT	67.60	pcf
UPWARD BOUYANT FORCE	-62.40	pcf

Horizontal Force = B(h) = (γ-(γ-62.4))*Ka)H^2/2, acts at HD/3:

Bouyant Load, WA: Vertical: Horizontal:

AREA #		VOLUME (CF)	GAMMA (#/CF)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WA	B1 (Ftg)	112.91	-62.40	-7.05	11.48	-80.85			
	B2 (Stem)	155.74	-62.40	-9.72	6.01	-58.36			
	B3 (Soil over Ftg)	233.44	-62.40	-14.57	17.71	-257.97			
	STATIC						0.00	7.29	0.00
	SEISMIC						0.00	7.29	0.00
TOTAL WA (Static)				**-31.33**		**-397.19**	**0.00**		**0.00**
TOTAL WA (Seismic)				**-31.33**		**-397.19**	**0.00**		**0.00**

Calculate Wind Loads

Primary Loads Section : 5.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WS	**Superstructure**				**0.40**	**24.53**	**9.81**
WL	**Superstructure**				**0.08**	**24.53**	**1.99**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Calculate Temperature Loads

Primary Loads Section : 6.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TU	**Superstructure**				**0.37**	**24.53**	**9.08**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Seismic Forces

Primary Loads Section : 7.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	Superstructure				2.150	24.53	52.74

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads:
(Ref: AASHTO 4th Ed., A11.1.1.1 for Mononobe-Okabe Analysis.)

GAMMA = unit weight of soil =	130.00	Lbs/CF		
H = height of soil face =	29.67	Feet		
PHI = angle of internal friction of soil =	33.00	Degrees =	0.58	Radians
DELTA = angle of friction between soil & abut =	22.00	Degrees =	0.38	Radians
i = backfill slope angle =	0.00	Degrees =	0.00	Radians
BETA = slope of wall to the vertical	0.00	Degrees =	0.00	Radians

A =	0.29			
kh = horizontal acceleration coefficient	0.145	Consider Cohesion?	N	------> kh = a * 0.5, Wall is NOT Restrained from Horizontal Movement
kv = vertical acceleration coefficient	0.000			
THETA = arc tan (kh/(1-Kv) =	8.25	Degrees =	0.14	Radians
Kae (per AASHTO Eq. A11.1.1.1-2) =	**0.363**	**<===== Governs.**		

<u>Earth Pressure Coefficients to be Used for Design per Geotechnical Report:</u>
Kae (geotech) = **0.000** **<=== Does not govern.**

N/A
NOT GIVEN IN

Load inclination from horizontal = δ =	22.00	degrees
Lateral EQ Load, Eae = 1/2*γ*Kae*H^2*(1-kv) =	20.77	klf
Arm for Horiz Load above BOF = H/3 =	9.89	ft (AASHTO pg 11-112)
Arm for Vert Load from Toe = BA =	22.95	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Eav = Eae*sin(δ) =	7.78	klf	Include EQ In Design =	Y
Horizontal Component, Eah = Eae*cos(δ) =	19.26	klf	EQ Factor =	1

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Seismic Forces

Primary Loads Section : 7.1

Include Seismic Passive Earth Pressure	Y	
Epe Factor	1	
kh = horizontal acceleration coefficient	0.145	
ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kpe = Seismic Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
Hff = Height of Soil at Front Face - 1'	8.87	ft
Lateral EQ Load, Epe = 1/2*γ*Kpe*H^2=	16.02	klf ---> Equation A11.4-4
Arm for Horiz Load above BOF = Hff/3 =	2.96	ft (AASHTO pg 11-112)

SECTION 11: WALLS, ABUTMENTS, AND PIERS 11-117

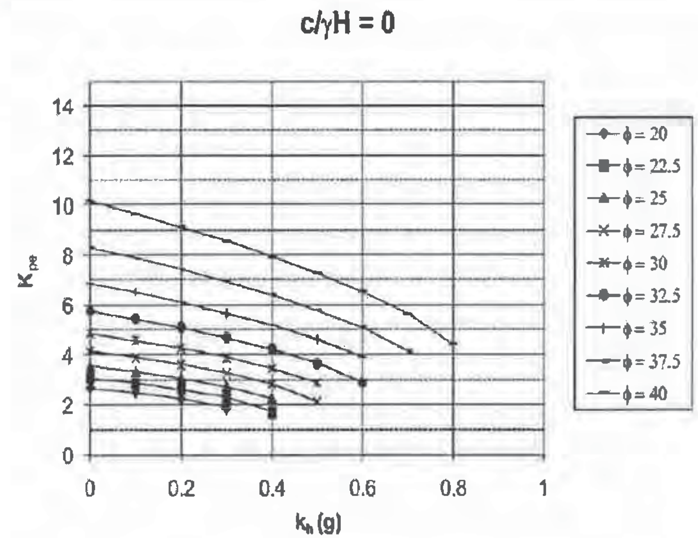


Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{h0}$ for wall heights greater than 20 ft.

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Calculate Seismic Forces Continued..

Primary Loads Section : 7.2

WALL INERTIA EFFECTS

Per **AASHTO DIV 1A 6.4.3,** seismic design should take into account forces arising from seismically induced lateral earth pressures (as computed above), additional forces arising from wall inertia and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely.

The following table computes the inertia forces due to the weight of the concrete and backfill.

kh = 0.145

AREA #		DL (Kips)	DL*kh (Kips)	ARM (Feet)	MOM (Ft x K)
DL Wall	D1	16.94	2.46	2.46	6.04
	D2	12.17	1.77	14.72	25.99
	D3	1.90	0.28	27.10	7.45
	D4	7.42	1.08	11.46	12.32
	Subtotal	**38.43**	**5.57**	**9.30**	**51.81**
DL Backfill	E1	6.43	0.93	17.99	16.77
	E2	3.37	0.49	27.10	13.25
	E3	29.53	4.28	17.29	74.06
	E4	2.53	0.37	7.40	2.72
	Subtotal	**41.87**	**6.07**	**17.59**	**106.80**
TOTAL		**80.30**	**11.64**	**13.62**	**158.61**

Total Seismic Loads, EQ:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	EQ Superstructure =				2.150	24.53	52.735
	Eae(v)	7.78	22.95	178.58			
	Eae(h)				19.26	9.89	190.46
	Epe(v)		22.95	0.00			
	Epe (h)				-16.02	2.96	-47.37
	Fwi(h)				11.64	13.62	158.61
TOTAL EQ		**7.78**		**178.58**	**17.03**		**354.43**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Notes for Khost Bridge **No 10**

Summary of Primary Loads

Load Combinations : 1.0

INCLUDE SEISMIC = Y

Load		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)	Notes	LRFD Load Combination Load Case
Dead Load	$DC_{SUB+SUPER}$	50.91	0.00	422.46	0.00	0.00	0.00	Super + Sub	
	DW	0.59	0.00	3.29	0.00	0.00	0.00	Super Only	
	DC_{SUB}	38.43	0.00	352.83	0.00	0.00	0.00	Sub Only - Construction	LC1 only
Earth Load	EH	3.95	22.95	90.73	4.32	12.85	153.75	All cases except Heel	Used in all load cases
	EH	7.76	22.95	178.13	19.21	9.89	189.98	For Heel Reinforcement	Not used in any load case
	EV	41.87	0.00	637.31	0.00	0.00	0.00		
Earth Load Surcharge	ES	3.56	0.00	56.32	2.69	0.00	39.84		
Live Load Surcharge	LS(v)	3.70	15.84	58.57	0.00	0.00	0.00		
	LS(h)	0.00	0.00	0.00	2.79	14.83	41.44		
Live Load	LL+IM+PED+BR+LS	8.91	0.00	87.66	3.28	0.00	53.40		
	LL+IM+PED+BR+LS	5.22	0.00	29.08	3.28	0.00	53.40	No LS for Sliding LC	LC4, LC8 & LC10
	LS	2.39	4.59	10.96	0.00	0.00	0.00		
Bouyant Load	WA	-31.33	0.00	-397.19	0.00	0.00	0.00	Static	
	WA	-31.33	0.00	-397.19	0.00	0.00	0.00	Seismic	LC9 &LC10
Wind Load	WS	0.00	0.00	0.00	0.40	24.53	9.81		
	WL	0.00	0.00	0.00	0.08	24.53	1.99		
Temperature Load	TU	0.00	0.00	0.00	0.37	24.53	9.08		
Seismic Load	EQ	7.78	0.00	178.58	17.03	0.00	354.43		
Vehicle Collision Load	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stem Wall	LC11 & LC12
	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stability	

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CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Limit States and Load Factors

Load Combinations : 2.0

Service Limit State

Per AASHTO 10.5.2, foundation design at the service limit state shall include settlements, horizontal movements, overall stability (of earth slopes) and scour at the design flood.
* These items are part of the geotechnical scope and are therefore NOT included in this design.

Strength Limit States

Per AASHTO 10.5.3, foundation design at the strength limit strength shall include structural resistance, scour, nominal bearing resistance, overturning or excessive loss of contact, sliding and constructability.
* These items, except scour, are addressed in this design.

Extreme Events Limit States

Per AASHTO 10.5.4, foundation shall be designed for extreme events such as a seismic event and vehicle collision.
* These items are addressed in this design.

Since these factors are 1.0, they have not yet been incorporated into the design template.

Computation of the Load Modification Factor, h_i:

		Extreme	Strength
h_D Ductility Factor, (AASHTO 1.3.3):		1.00	1.00
h_R Redundancy Factor, (AASHTO 1.3.4):		1.00	1.00
h_I Operational Importance Factor, (AASHTO 1.3.5):		1.00	1.00
h_i (for loads for which y_i(max) is appropriate) (AASHTO Eq 1.3.2.1-2):	$h_i = h_D h_R h_I \geq 0.95$	1.00	1.00
h_i (for loads for which y_i(min) is appropriate) (AASHTO Eq 1.3.2.1-3):	$h_i = 1 / h_D h_R h_I \leq 1.00$	1.00	1.00

h_D Ductility Factor (for all other limit states h_D = 1.00)

$h_D \geq$	1.05	for nonductile components and connections.
$h_D =$	1.00	for conventional designs and details complying with the specifications.
$h_D \geq$	0.95	for components and connections for which additional ductility-enhancing

h_R Redundancy Factor (for all other limit states h_R = 1.00)

$h_R \geq$	1.05	for nonredundant members
$h_R =$	1.00	for conventional levels of redundancy
$h_R \geq$	0.95	for exceptinal levels of redundancy

h_I Operational Importance Factor

$h_I \geq$	1.05	for a bridge of operational importance
$h_I =$	1.00	for typical bridges
$h_I \geq$	0.95	for relatively less important bridges

Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2), g_p:

	Maximum	Minimum	
DC (Dead Load, General):	1.25	0.90	
DW (Wearing Surface & Utilities):	1.50	0.65	
EH (Horiz Earth):	1.43	0.90	<-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.
ES (Horiz Earth):	1.50	0.75	
EV (Vertical Earth, Retaining Structure):	1.35	1.00	

Live Load Factor During a Seismic Event, g_{EQ}:

	Maximum	Minimum	
g_{EQ} (AASHTO C3.4.1):	0.50	0.00	**<— Seismic Included**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Load Combinations : 3.0

NOTES:

1. Load Combination Strength II does not need to be checked since it applies to special design vehicles.
2. Load Combination Strength III does not need to be checked during construction since WS is not a significant load.
3. Load Combination Strength IV does not need to be checked since it applies to bridges with very high dead load to live load ratios.
4. Load Combination Strength V does not need to be checked during construction since WS and WL are not significant loads.
5. Extreme Event load combinations do not need to be checked during construction.
6. Extreme Event II load combinations does not need to be checked for abutments.
7. Service limit state load combinations do not need to be checked for abutment stability / reinforcement.
8. Fatigue limit state load combinations do not need to be checked for abutment stability / reinforcement.
9. All remaining load cases shall be checked using load factors which would provide max effect for either bearing or sliding / eccentricity similar to AASHTO Figures C11.5.5-1 and C11.5.5.2.
10. Bouyancy has been included in sliding load combinations. A load factor of 0.0 has been used for bearing pressure load combinations since it is conservative to ignore sliding for these computations.

Strength	LC1	LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): gp max*(DCsub)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Strength	LC2	LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Bearing	LC3	LC3 - STRENGTH I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Sliding	LC4	LC4 - STRENGTH I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Bearing	LC5	LC5 - STRENGTH III BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Sliding	LC6	LC6 - STRENGTH III SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Bearing	LC7	LC7 - STRENGTH V BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Sliding	LC8	LC8 - STRENGTH V SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Extreme Bearing	LC9	LC9 - EXTREME EVENT I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+gEQ MAX*(LL+IM+PL+BR+LS)+1.0*(EQ)
Extreme Sliding	LC10	LC10 - EXTREME EVENT I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+gEQ MIN*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)
Extreme Bearing	LC11	LC11 - EXTREME EVENT II BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(CT)
Extreme Sliding	LC12	LC12 - EXTREME EVENT II SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(CT)

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

LRFD Load Combinations

Load Combinations : 3.1

LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): $q_{p\,max}$*(DCsub)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC_{SUB}	1.25	48.03		441.04	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.35	56.52		860.37	0.00		0.00
ES	1.50	5.33		84.48	4.03		59.77
SUM		**115.53**		**1515.19**	**10.18**		**278.87**

LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	63.64		528.08	0.00		0.00
DW	1.5	0.88		4.93	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.35	56.52		860.37	0.00		0.00
ES	1.50	5.33		84.48	4.03		59.77
SUM		**132.02**		**1607.15**	**10.18**		**278.87**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.2

LC3 - STRENGTH I BEARING: $g_{p,max}$*(DC+DW)+$g_{p,max}$*(EH)+$g_{p,max}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	63.64		528.08	0.00		0.00
DW	1.5	0.88		4.93	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.35	56.52		860.37	0.00		0.00
LL+IM+PL+BR+LS	1.75	15.60		153.40	5.74		93.46
WA	1.00	-31.33		-397.19	0.00		0.00
TU	0.50	0.00		0.000	0.1850		4.538
SUM		**110.95**		**1278.89**	**12.08**		**317.09**

LC4 - STRENGTH I SLIDING: $g_{p,min}$*(DC+DW)+$g_{p,max}$*(EH)+$g_{p,min}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	45.82		380.22	0.00		0.00
DW	0.65	0.38		2.14	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.00	41.87		637.31	0.00		0.00
LL+IM+PL+BR+LS	1.75	9.13		50.90	5.74		93.46
WA (static)	1.00	-31.33		-397.19	0.00		0.00
TU	0.50	0.00		0.00	0.185		4.538
SUM		**71.51**		**802.67**	**12.08**		**317.09**

LC5 - STRENGTH III BEARING: $g_{p,max}$*(DC+DW)+$g_{p,max}$*(EH)+$g_{p,max}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	63.64		528.08	0.00		0.00
DW	1.5	0.88		4.93	0.00		0.00
EH	1.425	5.63		129.29	6.15		219.10
EV	1.35	56.52		860.37	0.00		0.00
WA (static)	1.00	-31.33		-397.19	0.00		0.00
WS	1.40	0.00		0.00	0.56		13.74
TU	0.50	0.00		0.00	0.1850		4.5377
SUM		**95.35**		**1125.48**	**6.90**		**237.37**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.3

LC6 - STRENGTH III SLIDING: $q_{p\,min}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,min}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.90	45.82		380.22	0.00		0.00
DW	0.65	0.38		2.14	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.00	41.87		637.31	0.00		0.00
WA	1.00	-31.33		-397.19	0.00		0.00
WS	1.40	0.00		0.00	0.56		13.74
TU	0.50	0.00		0.00	0.1850		4.5377
SUM		**62.38**		**751.77**	**6.90**		**237.37**

LC7 - STRENGTH V BEARING: $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	63.64		528.08	0.00		0.00
DW	1.5	0.88		4.93	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.35	56.52		860.37	0.00		0.00
LL+IM+PL+BR+LS	1.35	12.03		118.34	4.43		72.09
WA	1.00	-31.33		-397.19	0.00		0.00
WS	0.40	0.00		0.00	0.16		3.92
WL	1.00	0.00		0.00	0.08		1.99
TU	0.50	0.00		0.00	0.1850		4.5377
SUM		**107.39**		**1243.83**	**11.01**		**301.64**

LC8 - STRENGTH V SLIDING: $q_{p\,min}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,min}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	45.82		380.22	0.00		0.00
DW	0.65	0.38		2.14	0.00		0.00
EH	1.425	5.63		129.29	6.15		219.10
EV	1	41.87		637.31	0.00		0.00
LL+IM+PL+BR+LS	1.35	7.04		39.26	4.43		72.09
WA	1.00	-31.33		-397.19	0.00		0.00
WS	0.40	0.00		0.00	0.16		3.92
WL	1.00	0.00		0.00	0.08		1.99
TU	0.50	0.00		0.00	0.1850		4.5377
SUM		**69.42**		**791.03**	**11.01**		**301.64**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC9 - EXTREME EVENT I BEARING: $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+$q_{EQ\,MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	63.64		528.08	0.00		0.00
DW	1.5	0.88		4.93	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.35	56.52		860.37	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.46		43.83	1.64		26.70
WA	0.00	0.00		0.00	0.00		0.00
EQ	1.00	7.78		178.58	17.03		354.43
SUM		**138.92**		**1745.08**	**24.83**		**600.23**

LC10 - EXTREME EVENT I SLIDING: $q_{p\,min}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,min}$*(EV)+$q_{EQ\,MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	45.82		380.22	0.00		0.00
DW	0.65	0.38		2.14	0.00		0.00
EH	1.43	5.63		129.29	6.15		219.10
EV	1.00	41.87		637.31	0.00		0.00
LL+IM+PL+BR+LS	0.00	0.00		0.00	0.00		0.00
WA (seismic)	1.00	-31.33		-397.19	0.00		0.00
EQ	1.00	7.78		178.58	17.03		354.43
SUM		**70.16**		**930.35**	**23.19**		**573.53**

Cantilever Abutment Design - Stability

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

References:
- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Notes for Khost Bridge No 10

Check Bearing Resistance (per AASHTO 11.6.3.2) -- ON SOIL

Stability : 1.0

If supported on soil, the vertical stress (σ_v) shall be calculated assuming a uniformly distributed pressure (V) over an effective base area (B-2e). AASHTO Fig 11.6.3.2-1 ---> $qr / \Phi\beta = q_n =$

If supported on rock, the vertical stress (σ_v) shall be calculated assuming a linearly distributed pressure over an effective base area. AASHTO Fig 11.6.3.2-2 ---> $qr / \Phi\beta = q_n =$

Factored Bearing Resistance, qr: $qr = \Phi\beta * q_n =$ 17.78 ksf **<— Note per Geotech, this is factored net bearing resistance**

Strength Bearing Resistance Factor, $\Phi\beta$ (AASHTO Table 11.5.7-1): 0.45 — $qr = \Phi\beta * q_n =$ 17.78 ksf ---> $qr / \Phi\beta = qn =$ 8.00 ksf

Extreme Event Bearing Resistance Factor, $\Phi\beta$ (AASHTO 10.5.5.3.3): 1.00 — $qr = \Phi\beta * q_n =$ 17.78 ksf ---> $qr / \Phi\beta = qn =$ 17.78 ksf

Note —> See AASHTO Table 11.5.7-1 to determine $\Phi\beta$ Factor

	LOAD COMBINATION	Vertical Force (Kips)	Resisting Moment (Ft x K)	Overturn Moment (Ft x K)	Mnet (Ft x K)	Eccentricity from Toe, et=Mnet/V (Ft)	Eccentricity from CL, e=B/2-et (Ft)	σ_v on soil (ksf)	$\sigma_{v\,max}$ on rock (ksf)	$\sigma_{v\,min}$ on rock (ksf)	$\sigma_v < \Phi\beta * q_n$; $\sigma_v <$ qr	
Strength	LC1	115.53	1515.19	278.87	1236.32	10.70	0.77	5.40	6.05	4.02	OK	
Strength	LC2	132.02	1607.15	278.87	1328.29	10.06	1.41	6.56	7.88	3.63	OK	
Bearing	LC3	110.95	1278.89	317.09	961.80	8.67	2.81	6.40	8.38	1.29	OK	
Sliding	LC4	71.51	802.67	317.09	485.58	6.79	4.68	5.27	7.02	0.00	OK	<--*N/A Sliding Combination
Bearing	LC5	95.35	1125.48	237.37	888.11	9.31	2.16	5.12	6.50	1.81	OK	
Sliding	LC6	62.38	751.77	237.37	514.40	8.25	3.23	3.78	5.01	0.42	OK	<--*N/A Sliding Combination
Bearing	LC7	107.39	1243.83	301.64	942.18	8.77	2.70	6.12	7.98	1.37	OK	
Sliding	LC8	69.42	791.03	301.64	489.39	7.05	4.43	4.92	6.56	0.00	OK	<--*N/A Sliding Combination
Ex. Bearing	LC9	138.92	1745.08	600.23	1144.85	8.24	3.23	8.43	11.17	0.94	OK	
Ex. Sliding	LC10	70.16	930.35	573.53	356.82	5.09	6.39	6.90	9.20	0.00	OK	<--*N/A Ex. Sliding Combination
Ex. Bearing	LC11	134.51	1522.67	245.80	1276.87	9.49	1.98	7.08	8.90	2.82	OK	
Ex. Sliding	LC12	70.20	751.77	219.10	532.67	7.59	3.89	4.63	6.17	0.00	OK	<--*N/A Ex. Sliding Combination

*** Sliding Load Combinations are Not Applicable for checking the Bearing**

Cantilever Abutment Design - Stability

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Check Overturning (per AASHTO 11.6.3.3) -- ON SOIL

Stability : 2.0

e allowable (ftgs on soil): 5.74 ft
e allowable (ftgs on rock): 8.61 ft
If e < e allowable, Overturning is OK:

	LOAD COMBINATION	Eccentricty from CL, e=B/2-et (Ft)	Check Overturning	
Strength	LC1	0.77	OK	
Strength	LC2	1.41	OK	
Bearing	LC3	2.81	OK	
Sliding	**LC4**	**4.68**	OK	<--*N/A **Sliding Combination**
Bearing	LC5	2.16	OK	
Sliding	**LC6**	**3.23**	**OK**	<--*N/A **Sliding Combination**
Bearing	LC7	2.70	OK	
Sliding	**LC8**	**4.43**	**OK**	<--*N/A **Sliding Combination**
Ex. Bearing	LC9	3.23	OK	
Ex. Sliding	**LC10**	**6.39**	N/A	<--*N/A **Ex. Sliding Combination**
Ex. Bearing	LC11	1.98	OK	
Ex. Sliding	**LC12**	**3.89**	OK	<--*N/A **Ex. Sliding Combination**

*** Sliding Load Combinations are Not Applicable for checking Overturning**

Cantilever Abutment Design - Stability

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Check Sliding (per AASHTO 10.6.3.4)

Stability : 3.0

Ignore Passive Resistance of Soil per MassHighway

Strength Sliding Resistance Factor, $\Phi\tau$ (AASHTO Table 11.5.7-1):	1.00	
Extreme Event Sliding Resistance Factor, $\Phi\varpi$ (AASHTO 10.5.5.3.3):	1.00	
Internal Friction Angle of Drained Soil, Φ_f:	33.00	degrees
tan δ: = tan Φ_f (per AASHTO 10.6.3.4-2):	0.65	for concrete against soil. Multiply by 0.8 for precast concrete footing

	LOAD COMBINATION	Vertical Force (Kips)	Rt = V * tan δ: (Kips)	Φτ (Strength) Φϖ (Extreme) (Kips)	Nom. Sliding Resistance Φτ*Rt (Kips)	Horiz Force (Kips)	Check Sliding	
Strength	**LC1**	115.53	75.02	1.00	75.02	10.18	OK	<--*N/A **Strength Combination**
Strength	**LC2**	132.02	85.73	1.00	85.73	10.18	OK	<--*N/A **Strength Combination**
Bearing	**LC3**	110.95	72.05	1.00	72.05	12.08	OK	<--*N/A **Bearing Combination**
Sliding	**LC4**	71.51	46.44	1.00	46.44	12.08	OK	
Bearing	**LC5**	95.35	61.92	1.00	61.92	6.90	OK	<--*N/A **Bearing Combination**
Sliding	**LC6**	62.38	40.51	1.00	40.51	6.90	OK	
Bearing	**LC7**	107.39	69.74	1.00	69.74	11.01	OK	<--*N/A **Bearing Combination**
Sliding	**LC8**	69.42	45.08	1.00	45.08	11.01	OK	
Ex. Bearing	**LC9**	138.92	90.22	1.00	90.22	24.83	OK	<--*N/A **Ex. Bearing Combination**
Ex. Sliding	**LC10**	70.16	45.56	1.00	45.56	23.19	OK	
Ex. Bearing	**LC11**	134.51	87.35	0.65	56.73	0.00	OK	<--*N/A **Ex. Bearing Combination**
Ex. Sliding	**LC12**	70.20	45.59	0.65	29.61	0.00	OK	

Cantilever Abutment Design - Stability

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Results Summary:

Stability : 4.0

STABILITY RESULTS:

LOAD COMBINATION:	BEARING RESISTANCE	OVERTURNING	SLIDING	
LC1	OK	OK	OK	<== Construction
LC2	OK	OK	OK	<== Construction
LC3	OK	OK	OK	
LC4	OK	OK	OK	
LC5	OK	OK	OK	
LC6	OK	OK	OK	
LC7	OK	OK	OK	
LC8	OK	OK	OK	
LC9	OK	OK	OK	
LC10	OK	N/A	OK	
LC11	OK	OK	OK	
LC12	OK	OK	OK	

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number: 1298\127-1298-12001-LT0077
Description: Khost Bridge No. 10
Structure: Abutment Design

Designed By: EWK
Checked By: SAM
Date: October 3, 2014

References:
AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
ACI 318-08 Building Code Requirements for Structural Concrete, 2005

Notes:
This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Notes for Khost Bridge No 10

Design Parameters

Reinforcement : 1.0

GEOMETRY

H of Footing, h :	4.92	ft
bw (per linear ft of wall) :	12.00	in

MATERIAL PROPERTIES

Compressive Strength: f'c :	4.00	ksi	
Min Yield Strength: fy :	60.00	ksi	
Max. Agg. Size :	1.50	in	
Es :	29000	ksi	AASHTO 5.4.3.2
Tension Reinforcement Strain: ε_s :	0.002	ε_s = fy / Es	
β :	1.881		AASHTO EQ 5.8.3.4.2-1

Design Heel and Toe Reinforcement

Reinforcement : 2.1

FACTORED HEEL DESIGN LOADS	Load Factor, γ_p AASHTO Table 3.4.1-2	Vertical Force & Design Shear (Kips)	Arm (Feet)	Design Moment (Ft x K)
DC (Heel Concrete)	1.25	8.47	4.59	38.87
EV (Heel Soil)	1.35	39.87125532	4.59	183.0090619
EH (Vertical Component)	1.43	11.06	9.18	101.53
LS	1.75	0.00	4.59	0.00
SUM		**59.40**		**323.41**

* See load combs, Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2) for the above Load Factors

* EH (Vertical Component) <-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.2

Footing Toe Width, BF: 3.94 ft

FACTORED TOE DESIGN LOADS LOAD COMBINATION	σ_v Factored Toe Pressure (ksf)	Factored Toe Shear (Kips)	Factored Toe Moment (Ft x K)
LC1	5.40	21.24	41.81
LC2	6.56	25.82	50.82
LC3	6.40	25.19	49.57
LC4	5.27	20.72	40.78
LC5	5.12	20.15	39.65
LC6	3.78	14.89	29.30
LC7	6.12	24.09	47.41
LC8	4.92	19.38	38.14
LC9	8.43	33.18	65.29
LC10	6.90	27.15	53.43
MAX		**33.18**	**65.29**

* Factored Toe Shear = Factored Toe Pressure * BF

* Factored Toe Moment = Factored Toe Shear * BF/2

Note: Based on AASHTO 10.6.5, the structural design of an eccentrically loaded foundation can assume a triangular or eccentrically loaded area. This spreadsheet conservatively assumes a uniform pressure of sv max over the toe of the footing. Based on AASHTO Figure C5.13.3.6.1-1, The toe shear can be computed at a distance dv from the face of support. This spreadsheet computes it at the support, which is conservative.

10.6.5—Structural Design

The structural design of footings shall comply with the requirements given in Section 5.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions.

FOOTING HEEL REINF (TOP BARS):

USE #	**8.00**	@	**4.00 IN**
Abar =	0.79	in^2	
dbar =	1.00	in	
Asprov =	2.37	in^2	

FOOTING TOE REINF (BOTTOM BARS):

USE #	**7.00**	@	**8.00 IN**
Abar =	0.60	in^2	
dbar =	0.88	in	
Asprov =	0.90	in^2	

CRITICAL SECTION FOR WALLS

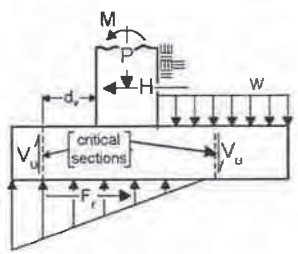

Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

CRITICAL SECTION FOR ABUTMENTS

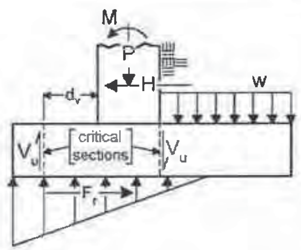

Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.3

CHECK FLEXURAL RESISTANCE	**HEEL**	**TOE**	**AASHTO 5.7, 5.7.2.2, 5.7.3.2, 5.7.3.2.2**	
Factored Moment, Mu =	323.41	65.29	k*ft	
Resistance Factor, phi: Φ =	0.90	0.90		AASHTO 5.5.4.2
Assume Cover, dc =	2.00	3.00	in	ACI 318-08 - 7.7
Shear Depth: ds =	56.54	55.60	in = h - cover - 1/2db(main)	
Depth of Equivalent Stress Block: a =	3.49	1.32	in = c*β1 = Asfy/0.85f'cb	AASHTO Eq. 5.7.3.1.1-4
Nominal Flexural resistance, Mn =	649.35	247.23	kip ft = [Asfy(ds-a/2)]/12	AASHTO Eq. 5.7.3.2.2-1
Factored Resistance, ΦMn =	584.41	222.51		AASHTO Eq. 5.7.3.2.1-1
As required for Mu:	0.67	0.17	in^2	
Flexure OK?	**OK**	**OK**		

CHECK MINIMUM REINFORCEMENT	**HEEL**	**TOE**	**AASHTO 5.7.3.3.2**	
Section Modulud: Sc =	6971.44	6971.44	in^3	
Compressive Strength: f'c =	4.00	4.00	ksi	
Modulus of Rupture: fr =	0.74	0.74	ksi = $0.37*(f'c)^{1/2}$	AASHTO 5.4.2.6
Cracking Moment: Mcr = Sc*fr =	429.91	429.91	kip ft	AASHTO Eq. 5.7.3.2.2-1
Factored Flexural Resistance: Mr1 = 1.2*Mcr =	515.89	515.89	kip ft	
Factored Moment, Mu =	323.41	65.29	k*ft	
Factored Flexural Resistance: Mr2 = 1.33*Mu =	430.14	86.84	kip ft	
Controlling Mr = min(Mr1, Mr2)	430.14	86.84	kip ft	
Factored Resistance, phi*Mn =	584.41	222.51		AASHTO Eq. 5.7.3.2.1-1
As required for Mr:	1.7295	0.3487	in^2	
As required for Temp Steel (#4@18"):	0.1333	0.1333	in^2	
As provided =	2.37	0.90	in^2	
Min Reinforcement OK?	**OK**	**OK**		

CHECK CRACK CONTROL BY DIST REINF.	**HEEL**	**TOE**	**AASHTO 5.7.3.4, 5.10.3.1**		
Exposure Factor: γ_e =	0.75	0.75	Class 2 Exposure		ASSHTO 5.7.3.4
β_s factor=	1.05	1.08		β_s factor = 1 + (dc / 0.7* (h-dc))	ASSHTO 5.7.3.4-1
f_{ss} =	36	36	ksi	f_{ss} = .6*fy	
s_{max} =	8.89	8.55	in	s_{max} < = 700 ge / βs fss	ASSHTO 5.7.3.4-1
s_{min} =	3.25	3.13	in	s_{min} = max(1.5*db,1.5*agg,1.5")+db	AASHTO 5.10.3.1.1
SPACING OK?	**OK**	**OK**			

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.4

CHECK SHEAR RESISTANCE	HEEL	TOE	AASHTO 5.13.3.6, 5.8.3		
Factored Shear Force, Vu =	59.40	33.18	kips		
Factored Moment, Mu =	323.41	65.29	k*in		
Es =	29000	29000			AASHTO 5.4.3.2
Resistance Factor, phi: Φ =	0.90	0.90			AASHTO 5.5.4.2
bw (per linear ft of wall) =	12.00	12.00	in		
Effective Depth: dv =	54.80	54.94	in	dv = max((ds-a/2),max(0.9ds, 0.72h))	AASHTO 5.8.2.9
H of Ftg, h:	59.04	59.04	in		
bw (per linear ft of ftg) =	12.00	12.00	in		
Area of Conc on Tension Side, Ac =	354.24	354.24	in	Ac = h*bw/2 =	
As (flexural) provd =	2.37	0.90	in^2		
Max. Size of Coarse Aggregate, ag =	1.50	1.50	in		
Mu min =	3254.97	1822.72	k*in	Mu min = Vu*dv =	
Mu (controlling) =	3254.97	1822.72	k*in		
Spg between top and bottom reinf, sx =	54.04	54.04	in		
Crack spg parameter, sxe =	35.01	35.01		sxe = 1.38*sx/(ag+0.63)	
Strain = ε_s=	0.0017	0.0025		ε_s=(Mu/dv+Vu)/(Es*As)	AASHTO EQ 5.8.3.4.2-4
Θ =	175.44	258.04		Θ = 29+3500*ε_s	AASHTO EQ 5.8.3.4.2-3
β =	1.44	1.14		β = 4.8/(1+750ε_s)*(51/(39+s_{xe}))	
Nom Shear Resistance, Vn1 =	657.57	659.29	kips	Vn1 = 0.25*f'c*bv*dv	AASHTO 5.8.3.3-2
Nominal Shear Resistance: Vn2 = Vc =	59.86	47.41	kips	Vn2 = Vc = 0.0316*β*f'c.5*bv*dv	AASHTO 5.8.3.3-3
Nom Shear Resistance, Vn =	59.86	47.41	kips	Vn = min (Vn1, Vn2)	
phi*Vn =	53.87	42.67			
Shear OK?	**OK**	**OK**			
Opposite Face Reinf As provd. =	0.90	2.37	in^2		
As min crack =	1.95	1.95	in^2	As min crack = 0.003*b*sx	
min (As front, back) > As min ?	**N/A**	**N/A**			

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CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

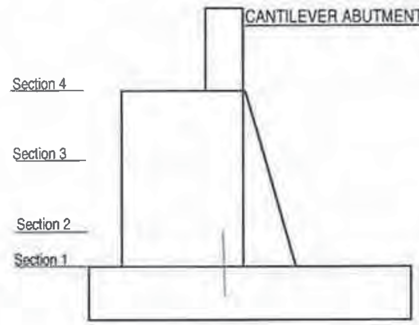
Stem Reinforcement

Reinforcement : 3.0

1. Reinforcement does not need to be checked for construction loading since that is a temporary load case.
Check the stem reinforcement at various locations along the stem and at the base of the backwall.

Height of Stem plus Backwall, h = H - F =	24.75	ft
Height of Backwall =	5.14	ft
Ftg Dowel Lap Length:	7.00	ft
Width of Stem at the Base:	9.18	ft
Width of Backwall:	2.46	ft
Width of Batter:	5.05	ft

Section	Height of h	Height from top	Width Batter	Width conc	
1	1.00	24.75	5.05	9.18	<=== This section is at the bottom of the stem.
2	0.72	17.75	3.24	7.38	<=== This section is at the top of the footing dowel.
3	0.46	11.44	1.62	5.76	<=== This section is halfway in between top of footing dowel and top of batter.
4	0.21	5.14		2.46	<=== This section is at the base of the backwall



Horizontal Earth Pressure, EH at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	29.668				4.32	12.85	153.75
Top of Ftg	24.748				3.01	8.25	24.79
Top of Dowel	17.748				1.55	5.92	9.14
Mid-Height	11.444				0.64	3.81	2.45
Bot of Backwall	5.14				0.13	1.71	0.22

Live Load Surcharge, LS at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	29.668				2.79	14.83	41.44
Top of Ftg	24.748				2.33	12.37	28.83
Top of Dowel	17.748				1.67	8.87	14.83
Mid-Height	11.444				1.08	5.72	6.17
Bot of Backwall	5.14				0.48	2.57	1.24

Seismic Load, EQ at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	29.668				17.03		354.43
Top of Ftg	24.748				14.21	12.37	175.83
Top of Dowel	17.748				10.19	8.87	90.43
Mid-Height	11.444				6.57	5.72	37.60
Bot of Backwall	5.14				2.95	2.57	7.58

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.1

Load Combination - STRENGTH I		At Top of Ftg		Top of Dowel		Mid-Height Abut		Bot of Backwall	
LOAD	Load Factor	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)
EH	1.43	4.28	35.32	2.20	13.03	0.92	3.49	0.18	0.32
LS	1.75	4.08	50.46	2.92	25.95	1.89	10.79	0.85	2.18
SUM		**8.36**	**85.78**	**5.13**	**38.98**	**2.80**	**14.28**	**1.03**	**2.49**

Load Combination - EXTREME EVENT I		At Top of Ftg		Top of Dowel		Mid-Height Abut		Bot of Backwall	
LOAD	Load Factor	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)
EH	1.43	4.28	35.32	2.20	13.03	0.92	3.49	0.18	0.32
LS	0.50	1.17	14.42	0.84	7.41	0.54	3.08	0.24	0.62
EQ	1.00	14.21	175.83	10.19	90.43	6.57	37.60	2.95	7.58
SUM		**19.66**	**225.57**	**13.23**	**110.87**	**8.03**	**44.17**	**3.38**	**8.52**

CHECK FLEXURAL RESISTANCE	**SECT 1**	**SECT 2**	**SECT 3**	**SECT 4**	**AASHTO 5.7**		
Section Height / Location =	24.75	17.75	11.44	5.14	ft		
Factored Moment, Mu =	225.57	110.87	44.17	8.52	k*ft		
Resistance Factor, phi: Φ =	0.90	0.90	0.90	0.90		AASHTO 5.5.4.2	
H of Stem, h:	9.18	7.38	5.76	2.46	ft		
Cover, dc =	2.00	2.00	2.00	2.00	in	ACI 318-08: Sec 7.7.1	<-- 2" for Concrete exposed to earth or weather: No. 6 thru No 18 bars
BAR # =	7.00	7.00	7.00	7.00			
SPACING =	8.00	8.00	8.00	8.00	in		
Main Abar =	0.60	0.60	0.60	0.60	in^2		
Main db =	0.875	0.875	0.875	0.875	in		
As provd. =	0.90	0.90	0.90	0.90	in^2		
Shear Depth: ds =	107.77	86.16	66.69	27.08	in. = h - cover - 1/2db(main)		
Depth of Equivalent Stress Block: a =	1.32	1.32	1.32	1.32	in = c*β1 = Asfy/0.85f'cb	AASHTO 5.7.2.2, 5.7.3.2	
Nominal Flexural resistance, Mn =	481.99	384.73	297.15	118.89	kip ft = [Asfy(ds-a/2)]/12	AASHTO 5.7.3.2.2	
Factored Resistance, phi*Mn =	433.79	346.26	267.43	107.00		AASHTO Eq. 5.7.3.2.1-1	
As required for Mu:	0.4695	0.2889	0.1489	0.0718	in^2		
Flexure OK?	**OK**	**OK**	**OK**	**OK**			

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CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number: 1298\127-1298-12001-LT0077
Description: Khost Bridge No. 10
Structure: Abutment Design

Designed By: EWK
Checked By: SAM
Date: October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.2

CHECK MINIMUM REINFORCEMENT	**SECT 1**	**SECT 2**	**SECT 3**	**SECT 4**	**AASHTO 5.7.3.3.2**	
Section Modulud: Sc =	24291.61	15698.29	9558.38	1742.86	in^3	
Modulus of Rupture: fr =	0.74	0.74	0.74	0.74	ksi = $0.37*(f'c)^{1/2}$	AASHTO 5.4.2.6
Cracking Moment: Mcr = Sc*fr =	1497.98	968.06	589.43	107.48	kip ft	
Factored Flexural Resistance: Mr1 = 1.2*Mcr =	1797.58	1161.67	707.32	128.97	kip ft	
Factored Moment, Mu =	225.57	110.87	44.17	8.52	k*ft	
Factored Flexural Resistance: Mr2 = 1.33*Mu =	300.01	147.46	58.75	11.34	kip ft	
Controlling Mr = min(Mr1, Mr2)	300.01	147.46	58.75	11.34	kip ft	
Factored Resistance, phi*Mn =	433.79	346.26	267.43	107.00		AASHTO Eq. 5.7.3.2.1-1
As required for Mr:	0.6213	0.3816	0.1962	0.0932	in^2	
As provided =	0.90	0.90	0.90	0.90	in^2	
Min Reinforcement OK?	**OK**	**OK**	**OK**	**OK**		

CHECK CRACK CONTROL BY DIST REINF.	**SECT 1**	**SECT 2**	**SECT 3**	**SECT 4**	**AASHTO 5.7.3.4, 5.10.3.1**	
Exposure Factor: γ_e =	0.75	0.75	0.75	0.75	Class 2 Exposure	ASSHTO 5.7.3.4
H of Stem, h:	110.21	88.60	69.13	29.52	in	
β_s factor=	1.03	1.03	1.04	1.10	1 + (dc / 0.7* (h-dc))	ASSHTO 5.7.3.4-1
f_{ss} =	36	36.00	36.00	36.00	ksi = .6*fy	
s_{max} =	10.21	10.12	9.99	9.21	in < = $700\ \gamma_e / \beta_s f_{ss}$	ASSHTO 5.7.3.4-1
Main db =	0.875	0.875	0.875	0.875	in	
s_{min} = max(1.5*db,1.5*agg,1.5")+db =	3.13	3.13	3.13	3.13	in	AASHTO 5.10.3.1.1
SPACING =	8.00	8.00	8.00	8.00	in	
SPACING OK?	**OK**	**OK**	**OK**	**OK**		

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.3

CHECK SHEAR TRANSFER	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.8.4.1, 5.8.4.3, 5.8.4.4
Cohesion Factor, c =	0.075	0.08	0.08	0.08	ksi, assumes CJ not intentionally roughened
Friction Factor, μ =	0.6	0.60	0.60	0.60	
Fraction of strength for interface shear, K1 =	0.2	0.20	0.20	0.20	
Limiting Interface Shear Resistance, K2 =	0.8	0.80	0.80	0.80	ksi
Lvi = H of Stem, h:	110.21	88.60	69.13	29.52	ft
bvi = bw (per linear ft of wall) =	12.00	12.00	12.00	12.00	in
Interface Area, Acv = Lvi*bvi =	1322.50	1063.14	829.58	354.24	in^2
Back Face (Flexural) As provd. =	0.90	0.90	0.90	0.90	in^2
Front Face (Dowels) As provd. =	0.44	0.44	0.44	0.44	in^2
Interface Reinf Provided, Avf = As back+front =	1.340	1.34	1.34	1.34	in^2
Vni = c*Acv+µ*Avf*Fy =	147.43	127.98	110.46	74.81	kips
Vni max1 = K1*f'c*Acv =	1058.00	850.52	663.66	283.39	kips
Vni max2 = K2*Acv =	1058.00	850.52	663.66	283.39	kips
Vni (controlling) =	147.43	127.98	110.46	74.81	kips
Fact. Interface Shear Resistance, Vri=phiVni =	132.68	115.18	99.41	67.33	kips
Fact. Interface Shear Load, Vui = Vu =	19.66	13.23	8.03	3.38	kips
Vu < Vri ?	**OK**	**OK**	**OK**	**OK**	
Min Interface Shear Reinf, Avf = 0.05*Acv/Fy =	1.102	0.886	0.691	0.295	in^2
Avf > Avfmin ?	**OK**	**OK**	**OK**	**OK**	

5.8.4.3—Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened

  c = 0.075 ksi
  μ = 0.6
  K_1 = 0.2
  K_2 = 0.8 ksi

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.4

CHECK SHEAR RESISTANCE	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.8.2, 5.8.3.3, 5.8.3.4.2	
Factored Shear Force, Vu =	19.66	13.23	8.03	3.38	kips	
Factored Moment, Mu =	2706.86	29.52	0.00	0.00	k*in	
Resistance Factor, phi: Φ =	0.90	0.90	0.90	0.90		AASHTO 5.5.4.2
Effective Depth: dv =	107.11	85.50	66.03	26.42	in=max((ds-a/2),max(0.9ds,0.72h))	AASHTO 5.8.2.9
H of Stem, h:	110.21	88.60	69.13	29.52	in	
Area of Conc on Tension Side, Ac = h*bw/2 =	661.25	531.57	414.79	177.12	in	
As (flexural, back face) provd =	0.90	0.90	0.90	0.90	in^2	
Max. Size of Coarse Aggregate, ag =	1.50	1.50	1.50	1.50	in	
Mu min = Vu*dv =	2105.42	1130.97	529.93	89.25	k*in	
Mu (controlling) =	2706.86	1130.97	529.93	89.25	k*in	
sx = dv	107.11	85.50	66.03	26.42	in ---> See Figure 5.8.3.4.2-3 (Case a)	
Crack spg parameter, sxe = 1.38*sx/(ag+0.63) =	69.39	55.39	42.78	17.12		
Strain = ε_s=(Mu/d+Vu)/(Es*As) =	0.0017	0.0010	0.0006	0.0003		
Θ = 29+35000*ε_s =	174.72	102.89	62.42	26.27		
β = 4.8/(1+750εs)*(51/(39+s_{xe}) =	0.99	1.47	2.05	3.65		
Nom Shear Resistance, Vn1 =	1285.30	1025.95	792.39	317.05	kips, Vn = 0.25*f'c*bv*dv	AASHTO 5.8.3.3-2
Nominal Shear Resistance: Vn2 = Vc =	80.07	95.53	102.59	73.20	kips, 0.0316*β*f'c$^{.5}$*bv*dv	AASHTO 5.8.3.3-3
Nom Shear Resistance, Vn = min (Vn1, Vn2) =	80.07	95.53	102.59	73.20	kips	
phi*Vn =	72.07	85.98	92.33	65.88		
Shear OK?	**OK**	**OK**	**OK**	**OK**		
Front Face (Dowels) As provd. =	0.44	0.44	0.44	0.44	in^2	
As min crack = 0.003*b*sx =	3.86	3.08	2.38	0.95	in^2 ---> Only Applicable for Figure 5.8.3.4.2-3 Case B	
min (As front, back) > As min ?	**N/A**	**N/A**	**N/A**	**N/A**		

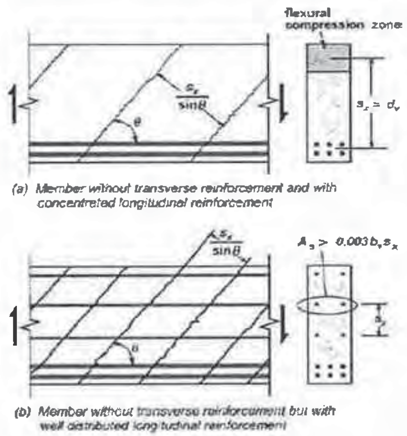


Figure 5.8.3.4.2-3—Definition of Crack Spacing Parameter, s_x

Crack Spacing Parameter, Sx --> Case = Case A

CANTILEVER ABUTMENT DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment Design	**Date:**	October 3, 2014

Results Summary:

Reinforcement : 4.0

REINFORCEMENT RESULTS:

= As Provided / As Required

		STIRUP #	BAR #	SPAC.	REINF. RATIO	FLEX OK?	LBS./ L.F.	LENGTH OF BAR	No. Bars per ft	Wt. of bar PER L.F.	As/LF	SHEAR OK?
A	TOE(bot):	---	7.00	8.00	2.58	OK	103.13	22.45	1.50	3.06	0.90	OK
B	HEEL(top):	---	8.00	4.00	1.37	OK	543.15	22.45	3.00	8.06	2.37	OK
C	STEM 1 (at top of ftg):	0.00	7.00	8.00	1.45	OK	45.94	10.00	1.50	3.06	0.90	OK
D	STEM 2 (at top of ftg dwl):	0.00	7.00	8.00	2.36	OK	45.94	10.00	1.50	3.06	0.90	OK
E	STEM 3 (midpt back face):	0.00	7.00	8.00	4.59	OK	45.94	10.00	1.50	3.06	0.90	OK
F	STEM 4 (at bot of bw):	0.00	7.00	8.00	9.65	OK	45.94	10.00	1.50	3.06	0.90	OK
G	STEM 5 (front face):	0.00	6.00	12.00	---	---	27.86	18.61	1.00	1.50	0.44	
H	STEM 6 (front face dowels):	0.00	6.00	12.00	---	---	3.37	2.25	1.00	1.50	0.44	
I	FOOTING (TOP):	0.00	6.00	6.00	---	---	137.45	1.00	45.90	2.99	0.88	
J	FOOTING (BOT.)	0.00	6.00	6.00	---	---	137.45	1.00	45.90	2.99	0.88	
K	STEM (longitudinal):	0.00	6.00	12.00	---	---	74.11	1.00	49.50	1.50	0.44	
					TOTAL WT. STEEL/FT OF ABUT.=		1210.25	LBS./LF				

SUBSTRUCTURE DESIGN

PIER DESIGN

* SUPERSTRUCTURE DEAD LOADS ON PIER
* SUPERSTRUCTURE LIVE LOADS ON PIER
* PIER STABILITY DESIGN

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/3/2014

References: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Notes: This spreadsheet computes the loads on an abutment, considering the spans left or right of the abutment is simply supported.

SUPERSTRUCTURE DEAD LOAD

Abutment Length =	10950	mm	35.92	ft
Pier Length =	12900	mm	42.31	ft
Use =	12900	mm	42.31	ft

Length =	33600	mm	110.21	ft
Spans =	2		2	
Span Length =	16800	mm	55.10	ft

Deck Width =	10950	mm	35.92	ft
Barrier =	225	mm	0.74	ft
Sidewalk =	1200	mm	3.94	ft
Barrier + Sidewalk =	1425	mm	4.67	ft
Roadway =	8100	mm	26.57	ft
No of Lanes	2		2.00	
Lane Width =	3657.6	mm	12.00	ft
Shoulder =	392.4	mm	1.29	ft

No of Beams =	6		6			
b =	600	mm	1.97	ft	23.62	in
d =	1500	mm	4.92	ft	59.04	in

Spacing =	1850	mm	6.07	ft	72.82	in
Distance for Beam to Beam =	9250	mm	30.34	ft	364.08	in
Overhang, OH =	850	mm	2.79	ft	33.46	in
Clear Overhang, OH_cl =	550	mm	1.80	ft	21.65	in
Clear Spacing bw Beams =	1250	mm	4.10	ft	49.20	in

Deck Thickness, ts =	225	mm	0.74	ft	8.86	in
Barrier Height =	1400	mm	4.59	ft	55.10	in
Sidewalk Height =	275	mm	0.90	ft	10.82	in

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/3/2014

SUPERSTRUCTURE DEAD LOAD

CAT		Width ft	Height ft	Length ft	Volume cf	Unit Weight lbs/cf	Weight Kips	Qty #	Total Kips	DC Kips	DW Kips
DC	Beams / Girders	1.97	4.92	55.10	533.55	150	80.03	6	480.19	480.19	
DC	Sidewalks	3.94	0.9	55.10	195.40	150	29.31	2	58.62	58.62	
DC	Safety Curbs	0	0	0	0.00	150	0.00	0	0.00	0.00	
DC	Barriers/ Rail	0.902	4.59	55.10	228.14	150	34.22	2	68.44	68.44	
DW	Wearing Surface	26.57	0.18	55.10	263.54	165	43.48	1	43.48		43.48
DC	End Diaphragms	4.1	4.92	2.46	49.62	150	7.44	10	74.43	74.43	
DC	Intermediate Diaphragms	4.1	4.1	0.98	16.47	150	2.47	5	12.36	12.36	
DW	Utilities				0.00	0	0.00	0	0.00		0.00
DW	Stay-In-Place Forms	0	0	0	0.00	0	0.00	0	0.00		0.00
DC	Deck	35.92	0.74	55.10	1460.59	150	219.09	1	219.09	219.09	
					0.00		0.00		0.00		
					0.00		0.00		0.00		
					0.00		0.00		0.00		
									956.62	913.13	43.48
										956.62	

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/3/2014

SUPERSTRUCTURE DEAD LOAD

		DC (kips)	DW (Kips)	DL (kips)
DC	Beams / Girders	480.19		
DC	Sidewalks	58.62		
DC	Safety Curbs	0.00	0.00	
DC	Barriers/ Rail	68.44		
DW	Wearing Surface		43.48	
DC	End Diaphragms	74.43		
DC	Intermediate Diaphragms	12.36		
DW	Utilities		0.00	
DW	Stay-In-Place Forms		0.00	
DC	Deck	219.09		

	DC	DW	DL	
Total Superstructure Dead Load	913.13	43.48	956.62	kips
Total Dead Load / Span End	456.57	21.74	478.31	kips / Span End
Length of Span Support Structure	42.31	42.31	42.31	
DL Per Span End (k/lf of Spprt. Structr.)	**10.79**	**0.51**	**11.30**	<-- Superstructure Dead Load (Input Tab)

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

References: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Notes: This spreadsheet computes the loads on an abutment, considering the spans left or right of the abutment is simply supported.

SUPERSTRUCTURE LOADING ON PIER - VERTICAL FORCES (CONT.)

Live Load, LL

Type of Truck: HL	93	
Roadway Width =	26.24	ft
Lane Width =	12	ft
Roadway / Lane Width =	2.19	
Use --> No of Lanes =	2	
Multiple Presence Factor, m =	1	

Table 3.6.1.1.2-1—Multiple Presence Factors, *m*

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65

Truck Loading:

	Left/Right Span		
Span Length, L =	55.10	ft	
Dynamic Load Allowance, (IM) =	1.33		
Number of Lanes =	2		
Multiple Presence Factor, m =	1.00		
Vmax =	59.1	kips / Lane	<-- T3.3.1.2 Shear & End Reactions
Vmax =	118.20	kips	<- Vmax * m * # of lanes
Reaction, LL V =	118.20	kips	
Reaction, (LL+IM) V =	157.2	kips	<-- IM * V
Total Reaction, Truck (LL) =	118.2	kips	
Total Reaction, Truck (LL+IM) =	157.2	kips	

Section 3.6.1.2.2

Section 3.6.2.1

Section 3.6.1.1.2

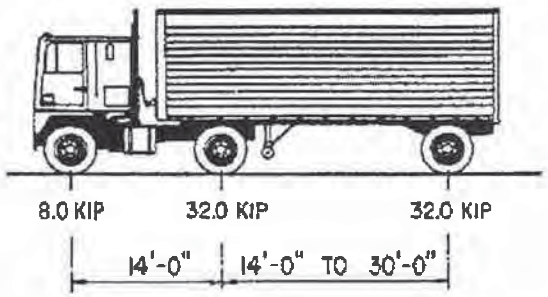


Figure 3.6.1.2.2-1

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

SUPERSTRUCTURE LOADING ON PIER - VERTICAL FORCES (CONT.)

Tandem Loading: Section 3.6.1.2.3

	Left/Right Span			
L =	55.10	ft		
Dynamic Load Allowance, (IM) =	1.33			Section 3.6.2.1
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
P1 =	25	kips		
P2 =	25	kips		
Axle Spacing =	4	ft		
Vmax =	48.19	kips/ Lane		
Vmax =	96.37	Kips	<- Vmax * m * # of lanes	
Reaction, LL V =	96.37	kips		
Reaction, (LL+IM) V =	128.17	kips	<-- IM * V	
Total Reaction, Tandem (LL) =	96.4	kips		
Total Reaction, Tandem (LL+IM) =	128.2	kips		

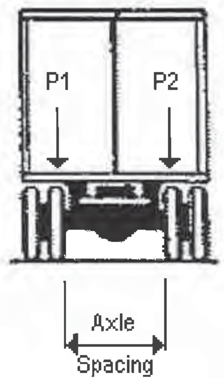
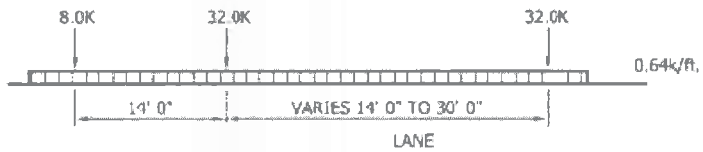


Figure 3.6.1.2.2-1

Live Load, LL (cont.)

Lane Loading: Section 3.6.1.2.4

	Left/Right Span			
L =	55.10	ft		
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
Lane Load =	0.64	klf		
Vmax =	17.63	kips/ Lane		
Vmax =	35.27	Kips	<- Vmax * m * # of lanes	
Reaction, Lane Load (LL) =	35.3	kips		
Total Reaction, Lane Load (LL) =	35.3	kips		



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BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

SUPERSTRUCTURE LOADING ON PIER - VERTICAL FORCES (CONT.)

Pedestrian Live Load

Pedestrian Live Load, PL =	0.075	ksf	<--- per AASHTO 3.6.1.6 for Sidewalks with a Width >= 2.0 ft
Width of Sidewalk =	3.95	ft	
PL =	0.296	klf	
Length of Sidewalk =	55.10	ft	Bridge Width = 42.31 ft
PL =	16.32	kips	--> PL / Span End= 16.32 kips; --> PL / LF of Abutment = 0.39 klf
Number of Sidewalks =	2.00		
Pedestrian Load Per Span =	32.65	kips	

Live Loads

	LL	IM	LL + IM
Truck	59.10	1.33	78.60
Tandem	48.19	1.33	64.09
Lane	17.63	1	17.63
Truck + Lane	76.73		96.24
Tandem + lane	65.82		81.72
Max	76.73		96.24

Max =	96.24	kips	
No of Lanes =	2.00		
m =	1.00		
LL+I =	192.47	kips	
Abutment Length =	42.31	ft	
LL+ I =	4.55	klf	
LL + I + PL =	**4.93**	**klf**	**<-- INPUT LOAD**

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BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier Design	Date:	10/10/2014

SUPERSTRUCTURE LOADING ON PIER - LATERAL FORCES

Braking Force, BR

Section 3.6.4

Notes: Dynamic Load Allowance increase not required. AASHTO3.6.2.1
Braking Force ONLY applies to fixed bearings
Braking Force includes multiple presence factor

25% Axle Weight of Design Truck =	25%	18.00	kips	Design Truck Axle Weight =	72
25% Axle Weight of Design Tandem =	25%	12.50	kips	Design Tandem Axle Weight =	50
5% (Axle Weight of Design Truck + Lane Load) =	5%	5.36	kips	Design Truck + Lane Axle Weight =	107.27
5% (Axle Weight of Design Tandem Load + Lane Load) =	5%	4.26	kips	Design Tandem + Lane Axle Weight =	85.27

Braking Force on Abutment (BR) =	18	kips	<---- 25% Axle Weight of Design Truck	
Number of Lanes =	2			
Multiple Presence Factor, m =	1			
Breaking Force Applied to 2 Abutements =	N			
BR =	**0.85**	**klf**	<--- BR / Abutment Length	**<-- Input Load**

Location of Load Application = *0.00* *ft above Bridge Seat*

SUPERSTRUCTURE LOADING ON PIER - LATERAL FORCES - EQ

EQ = 3.76 klf **<--- See Hand Calculations**

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB
ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD
GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/10/2014

AASHTO Standard Specifications for Highway Bridges - 17th edition 2002

Loading -- HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS— SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	485.3(b)	56.0(b)
2	16.0(b)	32.0(b)	44	520.9(b)	56.7(b)
3	24.0(b)	32.0(b)	46	556.5(b)	57.3(b)
4	32.0(b)	32.0(b)	48	592.1(b)	58.0(b)
5	40.0(b)	32.0(b)	50	627.9(b)	58.5(b)
6	48.0(b)	32.0(b)	52	663.6(b)	59.1(b)
7	56.0(b)	32.0(b)	54	699.3(b)	59.6(b)
8	64.0(b)	32.0(b)	56	735.1(b)	60.0(b)
9	72.0(b)	32.0(b)	58	770.8(b)	60.4(b)
10	80.0(b)	32.0(b)	60	806.5(b)	60.8(b)
11	88.0(b)	32.0(b)	62	842.4(b)	61.2(b)
12	96.0(b)	32.0(b)	64	878.1(b)	61.5(b)
13	104.0(b)	32.0(b)	66	914.0(b)	61.9(b)
14	112.0(b)	32.0(b)	68	949.7(b)	62.1(b)
15	120.0(b)	34.1(b)	70	985.6(b)	62.4(b)
16	128.0(b)	36.0(b)	75	1,075.1(b)	63.1(b)
17	136.0(b)	37.7(b)	80	1,164.9(b)	63.6(b)
18	144.0(b)	39.1(b)	85	1,254.7(b)	64.1(b)
19	152.0(b)	40.4(b)	90	1,344.4(b)	64.5(b)
20	160.0(b)	41.6(b)	95	1,434.1(b)	64.9(b)
21	168.0(b)	42.7(b)	100	1,524.0(b)	65.3(b)
22	176.0(b)	43.6(b)	110	1,703.6(b)	65.9(b)
23	184.0(b)	44.5(b)	120	1,883.3(b)	66.4(b)
24	192.7(b)	45.3(b)	130	2,063.1(b)	67.6
25	207.4(b)	46.1(b)	140	2,242.8(b)	70.8
26	222.2(b)	46.8(b)	150	2,475.1	74.0
27	237.0(b)	47.4(b)	160	2,768.0	77.2
28	252.0(b)	48.0(b)	170	3,077.1	80.4
29	267.0(b)	48.8(b)	180	3,402.1	83.6
30	282.1(b)	49.6(b)	190	3,743.1	86.8
31	297.3(b)	50.3(b)	200	4,100.0	90.0
32	312.5(b)	51.0(b)	220	4,862.0	96.4
33	327.8(b)	51.6(b)	240	5,688.0	102.8
34	343.5(b)	52.2(b)	260	6,578.0	109.2
35	361.2(b)	52.8(b)	280	7,532.0	115.6
36	378.9(b)	53.3(b)	300	8,550.0	122.0
37	396.6(b)	53.8(b)			
38	414.3(b)	54.3(b)			
39	432.1(b)	54.8(b)			
40	449.8(b)	55.2(b)			

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/10/2014

Table 3.3.1.1

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions

Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT. %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
1	8.0	6.3	0.1	8.1	0.50	32.0	25.0	0.3	32.3
2	16.0	12.5	0.3	16.3	0.50	32.0	25.0	0.6	32.6
3	24.0	18.8	0.7	24.7	0.50	32.0	25.0	1.0	33.0
4	32.0	25.0	1.3	33.3	0.50	32.0	25.0	1.3	33.3
5	40.0	31.3	2.0	42.0	0.50	32.0	30.0	1.6	33.6
6	48.0	37.5	2.9	50.9	0.50	32.0	33.3	1.9	35.3
7	56.0	43.8	3.9	59.9	0.50	32.0	35.7	2.2	38.0
8	64.0	50.0	5.1	69.1	0.50	32.0	37.5	2.6	40.1
9	72.0	62.5	6.5	78.5	0.50	32.0	38.9	2.9	41.8
10	80.0	75.0	8.0	88.0	0.50	32.0	40.0	3.2	43.2
11	84.5	92.0	9.3	101.3	0.40	32.0	40.9	3.5	44.4
12	92.2	104.0	11.1	115.1	0.40	32.0	41.7	3.8	45.5
13	103.0	115.9	13.4	129.3	0.45	32.0	42.3	4.2	46.5
14	110.9	128.3	15.5	143.8	0.45	32.0	42.9	4.5	47.3
15	118.8	140.6	17.8	158.4	0.45	34.1	43.3	4.8	48.1
16	126.7	153.0	20.3	173.3	0.45	36.0	43.8	5.1	48.9
17	134.6	165.4	22.9	188.3	0.45	37.6	44.1	5.4	49.6
18	142.6	177.8	25.7	203.4	0.45	39.1	44.4	5.8	50.2
19	150.5	190.1	28.6	218.7	0.45	40.4	44.7	6.1	50.8
20	158.4	202.5	31.7	234.2	0.45	41.6	45.0	6.4	51.4
21	166.3	214.9	34.9	249.8	0.45	42.7	45.2	6.7	52.0
22	174.2	227.3	38.3	265.6	0.45	43.6	45.5	7.0	52.5
23	182.2	239.6	41.9	281.5	0.45	44.5	45.7	7.4	53.0
24	190.1	252.0	45.6	297.6	0.45	45.3	45.8	7.7	53.5
25	198.0	264.4	49.5	313.9	0.45	46.1	46.0	8.0	54.1
26	210.2	276.8	53.5	330.3	0.45	46.8	46.2	8.3	55.1
27	226.1	289.1	57.7	346.9	0.45	47.4	46.3	8.6	56.0
28	241.9	301.5	62.1	363.6	0.45	48.0	46.4	9.0	57.0
29	257.8	313.9	66.6	380.5	0.45	48.8	46.6	9.3	58.1
30	273.6	326.3	71.3	397.5	0.45	49.6	46.7	9.6	59.2
31	289.4	338.6	76.1	414.7	0.45	50.3	46.8	9.9	60.2
32	307.0	351.0	81.1	432.1	0.45	51.0	46.9	10.2	61.2
33	324.9	363.4	86.2	449.6	0.45	51.6	47.0	10.6	62.2
34	332.0	375.0	92.5	467.5	0.50	52.2	47.1	10.9	63.1
35	350.0	387.5	98.0	485.5	0.50	52.8	47.1	11.2	64.0
36	368.0	400.0	103.7	503.7	0.50	53.3	47.2	11.5	64.9
37	386.0	412.5	109.5	522.0	0.50	53.8	47.3	11.8	65.7
38	404.0	425.0	115.5	540.5	0.50	54.3	47.4	12.2	66.5
39	422.0	437.5	121.7	559.2	0.50	54.8	47.4	12.5	67.2
40	440.0	450.0	128.0	578.0	0.50	55.2	47.5	12.8	68.0

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - SUPERSTRUCTURE DEAD LOAD

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	EWK
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment Design	Date:	10/10/2014

Table 3.3.1.2

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions

Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
42	485.2	474.6	139.7	624.9	0.45	56.0	47.6	13.4	69.4
44	520.9	499.5	153.3	674.2	0.45	56.7	47.7	14.1	70.8
46	556.5	524.3	167.6	724.1	0.45	57.4	47.8	14.7	72.1
48	592.2	549.0	182.5	774.6	0.45	58.0	47.9	15.4	73.4
50	627.8	573.8	198.0	825.8	0.45	58.6	48.0	16.0	74.6
52	663.4	598.5	214.2	877.6	0.45	59.1	48.1	16.6	75.7
54	699.1	623.3	230.9	930.0	0.45	59.6	48.1	17.3	76.8
56	734.7	648.0	248.4	983.1	0.45	60.0	48.2	17.9	77.9
58	770.4	672.8	266.4	1036.8	0.45	60.4	48.3	18.6	79.0
60	806.0	697.5	285.1	1091.1	0.45	60.8	48.3	19.2	80.0
62	841.6	722.3	304.4	1146.1	0.45	61.2	48.4	19.8	81.0
64	877.3	747.0	324.4	1201.7	0.45	61.5	48.4	20.5	82.0
66	912.9	771.8	345.0	1257.9	0.45	61.8	48.5	21.1	82.9
68	948.6	796.5	366.2	1314.8	0.45	62.1	48.5	21.8	83.9
70	984.2	821.3	388.1	1372.3	0.45	62.4	48.6	22.4	84.8
75	1070.0	887.5	450.0	1520.0	0.50	63.0	48.7	24.0	87.0
80	1160.0	950.0	512.0	1672.0	0.50	63.6	48.8	25.6	89.2
85	1250.0	1012.5	578.0	1828.0	0.50	64.1	48.8	27.2	91.3
90	1340.0	1075.0	648.0	1988.0	0.50	64.5	48.9	28.8	93.3
95	1430.0	1137.5	722.0	2152.0	0.50	64.9	48.9	30.4	95.3
100	1520.0	1200.0	800.0	2320.0	0.50	65.3	49.0	32.0	97.3
110	1700.0	1325.0	968.0	2668.0	0.50	65.9	49.1	35.2	101.1
120	1880.0	1450.0	1152.0	3032.0	0.50	66.4	49.2	38.4	104.8
130	2060.0	1575.0	1352.0	3412.0	0.50	66.8	49.2	41.6	108.4
140	2240.0	1700.0	1568.0	3808.0	0.50	67.2	49.3	44.8	112.0
150	2420.0	1825.0	1800.0	4220.0	0.50	67.5	49.3	48.0	115.5
160	2600.0	1950.0	2048.0	4648.0	0.50	67.8	49.4	51.2	119.0
170	2780.0	2075.0	2312.0	5092.0	0.50	68.0	49.4	54.4	122.4
180	2960.0	2200.0	2592.0	5552.0	0.50	68.3	49.4	57.6	125.9
190	3140.0	2325.0	2868.0	6028.0	0.50	68.5	49.5	60.8	129.3
200	3320.0	2450.0	3200.0	6520.0	0.50	68.6	49.5	64.0	132.6

http://www.dot.nd.gov/manuals/bridge/lrfd-bridge-design/Section03A.pdf

PIER DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 9, 2014

References:

AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012	AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011
ACI 318-08 Building Code Requirements for Structural Concrete, 2005	
2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions	

General Notes: This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Pier Options | Khost Bridge Notes

Project Notes: For the Preliminary Design it was assumed: 1) the centroid of the superstructure loads were located at the CL of the pier. 2) The superstructure Pier loads = 2 x the superstructure loads / pier length 3) This design was based on Pier 1

General Design Parameters

Input Section : 1.0

GEOMETRY INFORMATION INPUT:

PROPOSED TOP OF ROADWAY ELEV:			ft		m
PROPOSED TOP OF BACKWALL ELEV:		5969.02	ft	1819.823	m
PROPOSED BRIDGE SEAT ELEV:	H_Backwall = 0.00 ft	5969.02	ft	1819.823	m
PROPOSED TOP OF FOOTING ELEV:	H_Footing = 4.92 ft	5953.20	ft	1815.000	m
PROPOSED BOT. OF FOOTING ELEV:		5948.28	ft	1813.500	m
ELEVATION OF HIGH WATER:	FOR NO WATER = 0.00	5969.26	ft	1819.896	m
PROPOSED BRIDGE SEAT WIDTH:		4.92	ft	1.500	m
PROPOSED BACKWALL WIDTH:		0.00	ft	0.000	m
ABUTMENT/WALL DESIGN LENGTH:	1.00 Design for:	42.31	ft	12.900	m
FOOTING LENGTH		44.12	ft	13.450	m

DW CALCULATION INPUT:

WEARING SURFACE DEPTH:	2.00 IN x 1 Layers	0.17	ft	<---	0.051	m
ROADWAY WIDTH:		26.24	ft		8.000	m
BRIDGE SPAN:	Total Length = 110.208 <-- 2 Spans @	55.10	ft		16.800	m
NUMBER OF GIRDERS:		6				

Fill-in for Abutment / Pier Design
Ref: Khost Bridge No.10

MATERIAL PROPERTIES:

CUBIC WEIGHT CONCRETE:	150.00	pcf
COMP. STRENGTH OF CONC. = F'c:	4.00	ksi
MAXIMUM SIZE OF COARSE AGGREGATE	1.50	in
TENSILE STRENGTH OF REBAR = Fy:	60.00	ksi
CUBIC WEIGHT OF HOT MIX ASPHALT (HMA):	165.00	pcf

GEOTECHNICAL INFORMATION:

NOMINAL BEARING RESISTANCE (CAPACITY):	16.50	ksf	<-- Per Geotech Report
FACTORED BEARING RESISTANCE, qr :	7.43	ksf	<-- Per Geotech Report
WEIGHT OF SOIL BACKFILL:	130	Lbs/CF	<-- Per Geotech Report
WALL ON ROCK?	N	(Y OR N)	
WALL ON PILES?	N	(Y OR N)	
GRAVITY WALL?	N	(Y OR N)	
BETA: SLOPE OF BACKFILL:	0.00	DEG	<-- Per Civil
THETA: BATTER ANGLE BACKWALL:	90.00	DEG	AASHTO Table 3.11.5.3-1
PHI: FRICTION ANGLE OF BACKFILL:	33.00	DEG	<-- Per Geotech Report
DELTA: ANGLE BACKWALL FRICTION:	22.00	DEG	<-- Assumed δ=2/3 (ϕ)

CANTILEVER ABUTMENT DESIGN	N
GRAVITY ABUTMENT DESIGN	N
CANTILEVER WALL DESIGN	N
GRAVITY WALL DESIGN	N
PIER DESIGN	Y

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PIER DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

General Loading Parameters

Input Section : 2.0

LIVE LOAD INFORMATION:

APPROACH SLAB:	N	(Y OR N)
ROADWAY WITHIN H/2 OF TOP OF WALL:	N	(Y OR N)
Live Load Surcharge to be Considered?:	N	
SURCHARGE HEIGHT:	0.00	ft REF: Table 3.11.6.4-1
Construction Surcharge, q:	250.00	psf REF: C3.4.2.1

SEISMIC LOAD INFORMATION:

WALL RESTRAINED HORZ. MOVMT.(Y/N):	N	(Y OR N)
SEISMIC ACCELERATION COEFF. A:	0.290	REF: FIG.3.10.2.1-2, AASHTO
SEISMIC CATEGORY:	D	<---**Assumed based on Location & AASHTO Seimic Design Guide**

RAILING CLASS: S3-TL4 (CT) (PER MASSDOT LRFD BRIDGE MANUAL PART 1) 3.3.2.2 **<--- N/A**

Horizontal Railing Design Load	0.00	kips
Horizontal Railing Impact Length	0.00	ft
Wall Height+Rail Height	0.00	ft
Distributed Horizontal Railing Design Load @ top of wall	0.00	klf
Distributed Horizontal Railing Design Load @ bottom of wall	0.00	klf/wall height
Railing Dead Load	0.00	
Additional Moment From Railing Impact	0.00	<--- **Note:** The added moment from top of railing to bottom of railing is distributed along bottom of footing*

STREAM PRESSURE

Pmax	**0.00**	psf
Consider Stream Flow:	N	<--- Do not consider stream pressure perpendicular to the face of the pier since the Piers are parallel to the flow. Do not include stream pressure for this bridge.

SURCHARGE HEIGHT (Per ASSHTO 3.11.6.4 Live Load Surchage)

ABUTMENTS (N/A for PIERS) ---> Table 3.11.6.4-1

Table 3.11.6.4-1 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpindicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5	4
10	3
>20	2

Surcharge Height = 0.00 ft

RETAINING WALLS ---> Table 3.11.6.4-2

See Table 3.11.6.4-2 for Equivalent Height of Soil for Vechicular Loading on Retaining Walls Parallel to Traffic.

Retaining Wall Height (ft)	heq (ft) Distance from wall backface to edge of traffic.	
	0.0 ft	≥1.0 ft
5	5	2
10	3.5	2
>20	2	2

Distance from wall backface to edge of traffic = 0.0 ft

Surcharge Height = 0.00 ft

Note: See 3.11.6.5 for Possible Reduction of Surcharge

PIER DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Superstructure Loading Parameters

Input Section : 3.0

ADDITIONAL LOADS ON STRUCTURE

(load is per linear foot of structure (Abutment/ Pier/ Wall) NOT the Footing, arm from front edge of bridge seat)

LOADS		LOAD (klf)	ARM (feet)			
(DC+DW), SUPERSTRUCT. DEAD LOAD:	DL	22.61	2.46	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DC (Structural Components & nonstructural attachments)	DC	21.58	2.46	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DW (Wearing Surface & Utilities)	DW	1.03	2.46	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
(LL+IM+PL), LIVE LOAD, IMPACT AND PED LL:	LL+IM+PL	9.87	2.46	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
WS, WIND LOAD ON STRUCTURE:	WS	0.69	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
WL, WIND LOAD ON LIVE LOAD:	WL	0.14	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
BR, BREAKING LOAD :	BR	0.85	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
TU, THERMAL FORCE:	TU	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
EQ, SEISMIC LOAD ON SUPERSTRUCTURE:	EQ	3.76	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
CT, VEHICLE COLLISION LOAD	CT	0.00	0.00	Distance above top pf wall equal to the height of rail	Include =	Y

<u>Note</u>: Per AASHTO 11.5.1, abutments and retaining walls should be designed for EH, WA, LS, DS, DC, TU, EQ. Therefore, including wind and breaking forces is conservative. Say OK

PIER DESIGN -INPUT

General Information

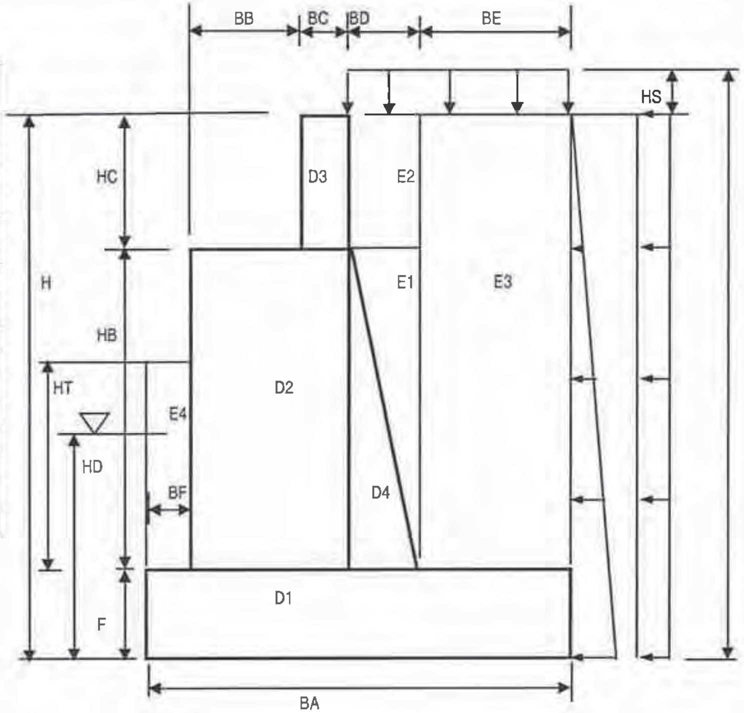
Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Abutment Geometry

Input Section : 4.0

CALCULATION OF WALL AND BACKFILL GEOMETRY:

		Prelim Size	User Adjust	Final Size (ft)	Approx Size (mm)
HEIGHT OF ABUTMENT / WALL, H:	H =	20.739	0.00	20.74	6400
HEIGHT OF FOOTING, F:	F =	4.920	0.00	4.92	1500
HEIGHT OF STEM, HB:	HB =	15.819	0.00	15.82	4900
HEIGHT OF BACKWALL, HC:	HC =	0.000	0.00	0.00	0
HEIGHT OF HIGH WATER, HD:	HD =	20.980	0.00	20.98	6400
HEIGHT OF SURCHARGE, HS:	HS =	0.000	0.00	0.00	0
WIDTH OF FOOTING, BA:	BA =	15.350	3.00	18.35	5600
WIDTH OF BRIDGE SEAT, BB:	BB =	4.920	0.00	4.92	1500
WIDTH OF BACKWALL, BC:	BC =	0.000	0.00	0.00	0
WIDTH OF BATTER OF STEM, BD:	BD =	0.000	0.00	0.00	0
WIDTH OF FOOTING HEEL, BE:	BE =	6.715	0.00	6.72	2050
WIDTH OF FOOTING TOE, BF:	BF =	6.715	0.00	6.72	2050
HEIGHT OF SOIL OVER TOE, HT:	HT =	1.640	0.00	1.64	500
HEIGHT OF SOIL OVER HEEL, HH:	HH =	1.640	0.00	1.64	500
HEIGHT OF SOIL AT BACKFACE FACE (HEEL), HS1	Hss1 =			6.56	2000
HEIGHT OF SOIL AT FRONT FACE FACE (TOE), HS2	Hss2 =			6.56	2000



OVERALL QUANTITIES:

WEIGHT OF CONCRETE WALL/L.F.:	25.217	Kips per l.f.
CONCRETE QUANTITY / L.F.:	6.226	C.Y. per l.f.

SUMMARY OF QUANTITIES:

STEEL / L.F. =	485.574	LBS/L.F.
CONC. / L.F =	6.226	C.Y./L.F.

Geometry Check: Check Width: **ok**
Check Height: **NG**

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

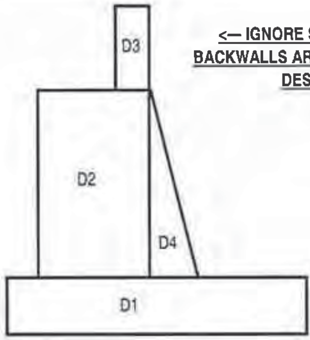
Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Pier Options **Khost Bridge Notes**

Calculate Dead Loads

Primary Loads Section : 1.0



<-- IGNORE SECTION D3, BACKWALLS ARE N/A FOR PIER DESIGN

Superstructure Loads:

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	Superstructure	21.58	9.18	198.01			
DW	Superstructure	1.03	9.18	9.43			

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads:

AREA #		Vertical: Volume (CF)	γ_{conc} (pcf)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	D1	90.28	150.00	13.54	9.18	124.25			
	D2	77.83	150.00	11.67	9.18	107.12			
	D3	0.00	150.00	0.00	11.64	0.00			
	D4	0.00	150.00	0.00	11.64	0.00			
	Subtotal Concrete			**25.22**		**231.37**			

<-- N/A FOR PIER DESIGN
<-- N/A, NO BATTER FOR THIS DESIGN

Total Dead Load:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	**46.80**		**429.37**			
TOTAL DW (Super)	**1.03**		**9.43**			
TOTAL DC (Substr. Only - Construction)	**25.22**		**231.37**			

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Earth Loads

Primary Loads Section : 2.0

Compute Horizontal Earth Pressure, EH:

Coulomb's Active Earth Pressure: (per MHD 3.1.5 and AASHTO 3.11.5.3)

PHI, $\phi f'$ =	33.00	Degrees,	Rad =	0.58
DELTA, δ =	22.00	Degrees,	Rad =	0.38
BETA, β =	0.00	Degrees,	Rad =	0.00
THETA, θ =	90.00	Degrees,	Rad =	1.57
Γ (per AASHTO Eq. 3.11.5.3-2)=	2.87			
Ka (per AASHTO Eq. 3.11.5.3-1)=	**0.264**			

Earth Pressure Coefficient to be Used for Design per MassDOT

All Walls on Rock	ko	0.455	
All Walls on Piles	ko	0.455	
Cantilever Walls < than 16' in Height	0.5*(Ko + Ka)	0.360	
Cantilever Walls > than 16' in Height	Ka	0.264	<-- USE
Gravity wall supported on Spread Footing	Ka	0.264	

At-Rest Earth Pressure Coeff:

Ko = **0.455**

Earth Pressure Coefficient to be Used for Design:

WALL ON LEDGE:	N	(Y OR N)
WALL ON PILES:	N	(Y OR N)
Wall Height:	20.73944	ft
Earth pressure Type:	Ka	
Ke =	**0.264**	<=== **Does not govern.**

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Ko =	0.49	
Ka =	0.32	
Ke (geotech) =	**0.320**	<===== **Governs.**

Compute Lateral Earth Pressure:

Application of lateral earth pressure shall be per AASHTO Figure C3.11.5.3-1. This shows a different application for Gravity and Cantilever (semi-gravity) walls.
Note that the reduction in lateral earth pressures due to the water table is not included in this section. It is included in the WA (Bouyancy) section of this design.

Cantilever (semi-gravity) Walls:

← ← THIS SECTION IS FOR CANTILEVER OR SEMI-GRAVITY WALLS ONLY ← ←

Load inclination from horizontal, min = $\phi/3$ =	11.00	degrees
Load inclination from horizontal, max = $\phi*2/3$ =	22.00	degrees
GAMMA =	130.00	pcf
H = Soil Height at Back face, Hss1	**6.56**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	**0.90**	kips
Arm for Horiz Load above BOF = H/3 =	2.19	ft
Arm for Vert Load from Toe = F =	18.35	ft

Consider minimum inclination for Sliding, Overturning and Bearing Pressure:

Vertical Component, Pav = Pa*sin(ϕ/3) =	0.17	klf
Horizontal Component, Pah = Pa*cos(ϕ/3) =	0.88	klf

Consider maximum inclination for Footing Heel Reinforcement:

Vertical Component, Pav = Pa*sin(ϕ*2/3) =	0.34	klf
Horizontal Component, Pah = Pa*cos(ϕ*2/3) =	0.83	klf

Gravity Walls:

← ← N/A ---> THIS SECTION IS FOR GRAVITY WALLS ONLY ← ←

Load inclination from horizontal = δ + (90−θ) =	22.00	degrees
GAMMA =	130.00	pcf
H =	**6.56**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	**0.90**	kips
Arm for Horiz Load above BOF = H/3 =	**2.19**	ft
Arm for Vert Load from Toe=(BF+BB+BC+BD*2/3) =	11.64	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Pav = Pa*sin(δ+(90−θ)) =	0.34	klf
Horizontal Component, Pah = Pa*cos(δ+(90−θ)) =	0.83	klf
Is the wall a Gravity Wall?	N	

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.1

Include Passive Earth Pressure	Y
Pp Factor	1

ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kp = Passive Earth Pressure Coefficient	**3.13**	Fig A11.4-2 **<u><-- For the preliminary design it was assumed kp = kpe (see below for kpe back-up)</u>**
γ = Unit Weight of Soil	130.00	pcf
H = Hss2= Height of Soil at Front Face	6.56	ft

Lateral EQ Load, Pp = 1/2*γ*Kp*H^2=	8.76	klf > Pah -------> Use Pp = Pah ---------> Pp = **0.88** klf
Arm for Horiz Load above BOF = H/3 =	2.19	ft (AASHTO pg 11-112)

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PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.2

Horizontal Earth Pressure, EH: Vertical: Horizontal:

AREA #	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EH: Pa	0.17	18.35	3.13	0.88	2.19	1.92
EH: Pp			0.00	-0.88	2.19	-1.92
EH (For all cases except heel reinforcement):	0.17	18.35	3.13	0.00	4.37	0.00
EH: Pa	0.34	18.35	6.15	0.83	2.19	1.81
EH: Pp			0.00			0.00
EH (For Heel Reinforcement):	0.34	18.35	6.15	0.83	2.19	1.81

<=== Note, Based on AASHTO Figure C11.5.6-1, both the vertical and horizontal components of EH should be included here because they carry the same load factor.

Vertical Earth Pressure, EV: Vertical: Horizontal:

AREA #		Volume (CF)	γ_{SOIL} (plf)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EV	E1	0.00	130.00	0.00	11.64	0.00			
	E2	0.00	130.00	0.00	11.64	0.00			
	E3	106.23	130.00	13.81	14.99	207.04			
	E4	11.01	130.00	1.43	3.36	4.81			
TOTAL EV				**15.24**		**211.85**			

<-- **N/A Batter = 0** (E1)
<-- **N/A Batter = 0** (E2)

Note, **per AASHTO 11.6.1.2**, the weight of the soil over the battered portion of the stem or over the base of a footing may be considered as part of the effective weight of the abutment. This is consistant with design.

Earth Surcharge, ES: (This applies for construction case only)

q =	250.00	psf
Uniform Load on Wall, p=Ke*q =	0.080	ksf
Wall Height, H =	20.74	Feet
Heel Length, BE =	6.72	Feet
Footing Width, BA =	18.35	Feet
Wall Length Considered =	1.00	ft

Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
ES	$P_{con}(h)$ = p*H*Length =				1.66	10.37	17.20
	$P_{con}(v)$ = q*BE*Length =	1.68	14.99	25.17			
TOTAL ES		**1.68**		**25.17**	**1.66**		**17.20**

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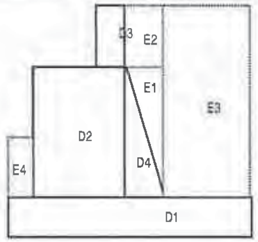
PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Live Loads

Primary Loads Section : 3.0



<--- IGNORE SECTION D3, BACKWALLS ARE N/A FOR PIER DESIGN

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LL+IM+PL	**Superstructure**	**9.87**	**9.18**	**90.55**			
BR	**Superstructure**				**0.851**	**20.74**	**17.65**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Live Load Surcharge Loads: LS

Per AASHTO 3.11.6.4, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall.
If the surcharge is for highway, the intensity of the load shall be consistent with provisions of Article 3.6.1.2. See Tables 3.11.6.4-1 and 3.11.6.4-2 for equivalent heights.

Compute Horizontal Live Load Surcharge: (To be used for bearing pressure and sliding load cases):

Ke =	0.264	
Unit Weight of Soil, γ =	130.000	pcf
Surcharge Height, heq =	0.00	Feet
LS(h) =(Ke)(γ)(heq)*H =	0.00	kips
Moment arm = H/2 =	10.37	kips

Compute Vertical Live Load Surcharge: (To be used for bearing pressure cases only):

LS(v) =(γ)(heq)(BD+BE) =	0.00	kips
Moment arm = Ba-(BD+BE)/2 =	14.99	kips

Compute Vertical Live Load Surcharge: (To be used for heel reinf cases only):

LS(v) =(γ)(heq)(BE) =	0.00	kips
Moment arm (to back of batter) = BE/2 =	3.36	kips

Live Load Surcharge, LS: Summary Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LS	**LS(v)**	**0.00**	**14.99**	**0.00**			
	LS(h)				**0.00**	**10.37**	**0.00**

Total Live Load Load: Vertical: Horizontal:

AREA #	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL LL+IM+PED+BR+LS	**9.87**		**90.55**	**0.85**		**17.65**
TOTAL LL+IM+PED+BR+LS (Sliding Only)	**9.87**		**90.55**	**0.85**		**17.65**
TOTAL LS (Heel Reinf Only)	**0.00**	**3.36**	**0.00**			

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Water load (Buoyancy Forces)

Primary Loads Section : 4.0

HEIGHT OF STEM AT HIGH WATER:	16.06	
HEIGHT OF FOOTING AT HIGH WATER:	4.92	
WIDTH OF FOOTING, BA	18.35	
SOIL WEIGHT - WATER WEIGHT	67.60	pcf
UPWARD BOUYANT FORCE	-62.40	pcf

Horizontal Force = B(h) = (γ-(γ-62.4))*Ka)H^2/2, acts at HD/3:

INCLUDE HORIZONTAL FORCE? N

<-- Note: The Horizontal load is Not Applicable since the hydrostatic force is equal and opposite on both sides.

Bouyant Load, WA: Vertical: Horizontal:

AREA #		VOLUME (CF)	GAMMA (#/CF)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WA	B1 (Ftg)	90.28	-62.40	-5.63	9.18	-51.69			
	B2 (Stem)	79.02	-62.40	-4.93	9.18	-45.24			
	B3 (Soil over Ftg)	215.69	-62.40	-13.46	14.99	-201.78			
	STATIC						0.00	6.99	0.00
	SEISMIC						0.00	6.99	0.00
TOTAL WA (BL) (Static)				**-24.02**		**-298.71**	**0.00**		**0.00**
TOTAL WA (BL) (Seismic)				**-24.02**		**-298.71**	**0.00**		**0.00**

Calculate Stream Flow Pressure

Primary Loads Section : 4.1

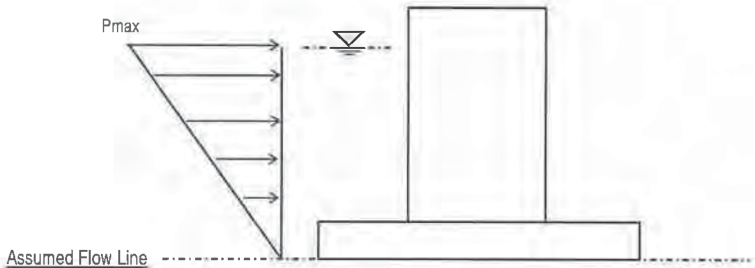
Note: The flow line is conservatively assumed to act at the bottom of the footing

Pmax:	0.0000	ksf
APPLIED:	N	

Force = 0.5 * Pmax * HD

Arm = HD * (2/3)

LOAD	HORIZONTAL FORCE (Kips)	ARM (Feet)	MOM (Ft x K)
WA (SF)	0.00	13.99	0.00



Calculate Water Load & Stream Flow Load WA

Primary Loads Section : 4.2

Water Load (Bouyancy) & Stream Flow, WA: Vertical: Horizontal:

AREA #	VOLUME (CF)	GAMMA (#/CF)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL WA (Static)			-24.02		-298.71	0.00		0.00
TOTAL WA (Seismic)			-24.02		-298.71	0.00		0.00

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PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Wind Loads

Primary Loads Section : 5.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WS	**Superstructure**				**0.69**	**20.74**	**14.23**
WL	**Superstructure**				**0.14**	**20.74**	**2.95**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Calculate Temperature Loads

Primary Loads Section : 6.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TU	**Superstructure**				**0.00**	**20.74**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Seismic Forces

Primary Loads Section : 7.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	**Superstructure**				**3.760**	**20.74**	**77.98**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads:
(Ref: AASHTO 4th Ed., A11.1.1.1 for Mononobe-Okabe Analysis.)

GAMMA = unit weight of soil =	130.00	Lbs/CF		
H = height of soil face =	20.74	Feet		
PHI = angle of internal friction of soil =	33.00	Degrees =	0.58	Radians
DELTA = angle of friction between soil & abut =	22.00	Degrees =	0.38	Radians
i = backfill slope angle =	0.00	Degrees =	0.00	Radians
BETA = slope of wall to the vertical	0.00	Degrees =	0.00	Radians

A = 0.29

kh = horizontal acceleration coefficient 0.435 Consider Cohesion? N ------> kh = a * 0.5, Wall is NOT Restrained from Horizontal Movement

kv = vertical acceleration coefficient 0.000

THETA = arc tan (kh/(1-Kv) = 23.51 Degrees = 0.41 Radians

Kae (per AASHTO Eq. A11.1.1.1-2) = **0.731** **<===== Governs.**

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Kae (geotech) = 0.000 **<=== Does not govern.**

N/A
NOT GIVEN IN
GEOTECH REPORT

Load inclination from horizontal = δ =	22.00	degrees
Lateral EQ Load, Eae = 1/2*γ*Kae*H^2*(1-kv) =	20.44	klf
Arm for Horiz Load above BOF = H/3 =	6.91	ft (AASHTO pg 11-112)
Arm for Vert Load from Toe = BA =	18.35	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Eav = Eae*sin(δ) = 7.66 klf Include EQ In Design = Y

Horizontal Component, Eah = Eae*cos(δ) = 18.95 klf EQ Factor = 1

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PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Seismic Forces

Primary Loads Section : 7.1

Include Seismic Passive Earth Pressure	Y	
Epe Factor	1	
kh = horizontal acceleration coefficient	0.435	
∅ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ∅ --> 11.6.5.5
Kpe = Seismic Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
Hff = Height of Soil at Front Face	6.56	ft
Lateral EQ Load, Epe = 1/2*γ*Kpe*H^2=	8.76	klf ---> Equation A11.4-4
Arm for Horiz Load above BOF = Hff/3 =	2.19	ft (AASHTO pg 11-112)

SECTION 11: WALLS, ABUTMENTS, AND PIERS 11-117

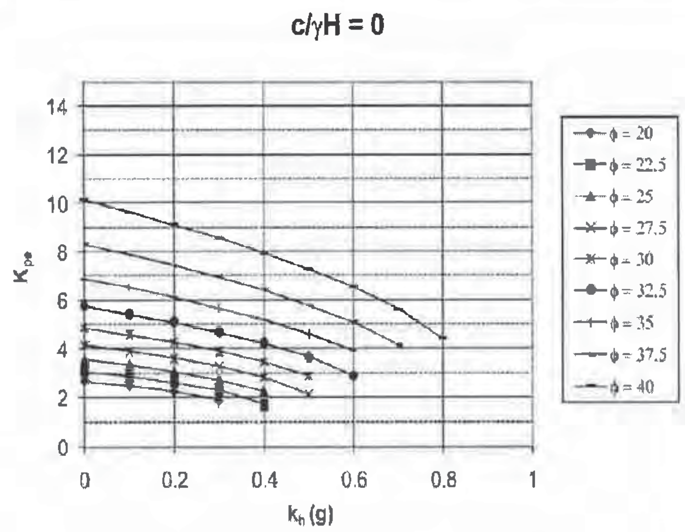

Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{h0}$ for wall heights greater than 20 ft

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PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Seismic Forces Continued..

Primary Loads Section : 7.2

WALL INERTIA EFFECTS

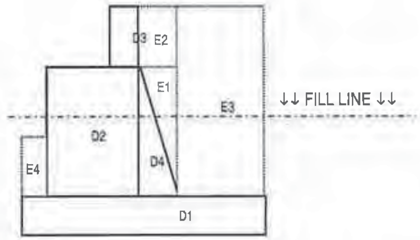
Per **AASHTO DIV 1A 6.4.3**, seismic design should take into account forces arising from seismically induced lateral earth pressures (as computed above), additional forces arising from wall inertia and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely.

The following table computes the inertia forces due to the weight of the concrete and backfill.

kh = 0.435

AREA #		DL (Kips)	DL*kh (Kips)	ARM (Feet)	MOM (Ft x K)	% of DL to be included
DL Wall	D1	0.00	0.00	2.46	0.00	0%
	D2	10.46	4.55	12.83	58.40	90%
	D3	0.00	0.00	20.74	0.00	100%
	D4	0.00	0.00	10.19	0.00	0% n/a
	Subtotal	**10.46**	**4.55**	**12.83**	**58.40**	
DL Backfill	E1	0.00	0.00	15.47	0.00	0%
	E2	0.00	0.00	20.74	0.00	0%
	E3	0.00	0.00	12.83	0.00	0%
	E4	0.00	0.00	5.74	0.00	0%
	Subtotal	**0.00**	**0.00**	**0.00**	**0.00**	
TOTAL		**10.46**	**4.55**	**12.83**	**58.40**	

FOR PIERS: Include DL above Fill Only



Total Seismic Loads, EQ:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	EQ Superstructure =				3.760	20.74	77.980
	Eae(v)	7.66	18.35	140.49			
	Eae(h)				0.00	6.91	0.00
	Epe(v)		18.35	0.00			
	Epe				0.00	2.19	0.00
	Fwi(h)				4.55	12.83	58.40
TOTAL EQ		**7.66**		**140.49**	**8.31**		**136.38**

% Eae(h) to be included:

0% **FOR PIERS: M-O ANALYSIS IS FOR RETAINED SOILS --> N/A FOR PIERS**

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PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Calculate Vehicle Collision Loads

Primary Loads Section : 8.2

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
CT (Stem Design)	**Superstructure**				0.00	0.00	0.00
CT	**Superstructure**				0.00	0.00	0.00

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 10, 2014

Summary of Primary Loads

Primary Loads Section : 9.2

	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	46.80		429.37			
TOTAL DW (Super)	1.03		9.43			
TOTAL DC (Substr. Only - Construction)	25.22		231.37			
EH (For all cases except heel reinforcement):	0.17	18.35	3.13	0.00	4.37	0.00
EH (For Heel Reinforcement):	0.34	18.35	6.15	0.83	2.19	1.81
TOTAL EV	15.24		211.85			
TOTAL ES	1.68		25.17	1.66		17.20
TOTAL LL+IM+PED+BR+LS	9.87	0.00	90.55	0.85	0.00	17.65
TOTAL LL+IM+PED+BR+LS (Sliding Only)	9.87	0.00	90.55	0.85	0.00	17.65
TOTAL LS (Heel Reinf Only)	0.00	3.36	0.00	0.00	0.00	0.00
TOTAL WA (Static)	-24.02		-298.71	0.00		0.00
TOTAL WA (Seismic)	-24.02		-298.71	0.00		0.00
WS Superstructure				0.69	20.74	14.23
WL Superstructure				0.14	20.74	2.95
TU Superstructure				0.00	20.74	0.00
TOTAL EQ	7.66		140.49	8.31		136.38
CT (Stem Design)	0.00	0.00	0.00	0.00	0.00	0.00
CT	0.00	0.00	0.00	0.00	0.00	0.00

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Khost Bridge Notes

Summary of Primary Loads

Load Combinations : 1.0

INCLUDE SEISMIC = Y

Load		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)	Notes	LRFD Load Combination Load Case
Dead Load	$DC_{SUB+SUPER}$	46.80	0.00	429.37	0.00	0.00	0.00	Super + Sub	
	DW	1.03	0.00	9.43	0.00	0.00	0.00	Super Only	
	DC_{SUB}	25.22	0.00	231.37	0.00	0.00	0.00	Sub Only - Construction	LC1 only
Earth Load	EH	0.17	18.35	3.13	0.00	4.37	0.00	All cases except Heel	Used in all load cases
	EH	0.34	18.35	6.15	0.83	2.19	1.81	For Heel Reinforcement	Not used in any load case
	EV	15.24	0.00	211.85	0.00	0.00	0.00		
Earth Load Surcharge	ES	1.68	0.00	25.17	1.66	0.00	17.20		
Live Load Surcharge	LS(v)	0.00	14.99	0.00	0.00	0.00	0.00		
	LS(h)	0.00	0.00	0.00	0.00	10.37	0.00		
Live Load	LL+IM+PED+BR+LS	9.87	0.00	90.55	0.85	0.00	17.65		
	LL+IM+PED+BR+LS	9.87	0.00	90.55	0.85	0.00	17.65	No LS for Sliding LC	LC4, LC8 & LC10
	LS	0.00	3.36	0.00	0.00	0.00	0.00		
Bouyant Load & Stream Force	WA	-24.02	0.00	-298.71	0.00	0.00	0.00	Static	
	WA	-24.02	0.00	-298.71	0.00	0.00	0.00	Seismic	LC9 &LC10
Wind Load	WS	0.00	0.00	0.00	0.69	20.74	14.23		
	WL	0.00	0.00	0.00	0.14	20.74	2.95		
Temperature Load	TU	0.00	0.00	0.00	0.00	20.74	0.00		
Seismic Load	EQ	7.66	0.00	140.49	8.31	0.00	136.38		
Vehicle Collision Load	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stem Wall	LC11 & LC12
	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stability	

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PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Limit States and Load Factors

Load Combinations : 2.0

Service Limit State

Per AASHTO 10.5.2, foundation design at the service limit state shall include settlements, horizontal movements, overall stability (of earth slopes) and scour at the design flood.
* These items are part of the geotechnical scope and are therefore NOT included in this design.

Strength Limit States

Per AASHTO 10.5.3, foundation design at the strength limit strength shall include structural resistance, scour, nominal bearing resistance, overturning or excessive loss of contact, sliding and constructability.
* These items, except scour, are addressed in this design.

Extreme Events Limit States

Per AASHTO 10.5.4, foundation shall be designed for extreme events such as a seismic event and vehicle collision.
* These items are addressed in this design.

Since these factors are 1.0, they have not yet been incorporated into the design template.

Computation of the Load Modification Factor, h_i:

		Extreme	Strength
h_D Ductility Factor, (AASHTO 1.3.3):		1.00	1.00
h_R Redundancy Factor, (AASHTO 1.3.4):		1.00	1.00
h_I Operational Importance Factor, (AASHTO 1.3.5):		1.00	1.00
h_i (for loads for which γ_i(max) is appropriate) (AASHTO Eq 1.3.2.1-2):	$h_i = h_D h_R h_I \geq 0.95$	1.00	1.00
h_i (for loads for which γ_i(min) is appropriate) (AASHTO Eq 1.3.2.1-3):	$h_i = 1 / h_D h_R h_I \leq 1.00$	1.00	1.00

h_D Ductility Factor (for all other limit states $h_D = 1.00$)

$h_D \geq$	1.05	for nonductile components and connections.
$h_D =$	1.00	for conventional designs and details complying with the specifications.
$h_D \geq$	0.95	for components and connections for which additional ductility-enhancing

h_R Redundancy Factor (for all other limit states $h_R = 1.00$)

$h_R \geq$	1.05	for nonredundant members
$h_R =$	1.00	for conventional levels of redundancy
$h_R \geq$	0.95	for exceptinal levels of redundancy

h_I Operational Importance Factor

$h_I \geq$	1.05	for a bridge of operational importance
$h_I =$	1.00	for typical bridges
$h_I \geq$	0.95	for relatively less important bridges

Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2), g_p:

	Maximum	Minimum	
DC (Dead Load, General):	1.25	0.90	
DW (Wearing Surface & Utilities):	1.50	0.65	
EH (Horiz Earth):	1.43	0.90	<-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.
ES (Horiz Earth):	1.50	0.75	
EV (Vertical Earth, Retaining Structure):	1.35	1.00	

Live Load Factor During a Seismic Event, g_{EQ}:

	Maximum	Minimum	
g_{EQ} (AASHTO C3.4.1):	0.50	0.00	**<--- Seismic Included**

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Load Combinations : 3.0

NOTES:

1. Load Combination Strength II does not need to be checked since it applies to special design vehicles.
2. Load Combination Strength III does not need to be checked during construction since WS is not a significant load.
3. Load Combination Strength IV does not need to be checked since it applies to bridges with very high dead load to live load ratios.
4. Load Combination Strength V does not need to be checked during construction since WS and WL are not significant loads.
5. Extreme Event load combinations do not need to be checked during construction.
6. Extreme Event II load combinations does not need to be checked for abutments.
7. Service limit state load combinations do not need to be checked for abutment stability / reinforcement.
8. Fatigue limit state load combinations do not need to be checked for abutment stability / reinforcement.
9. All remaining load cases shall be checked using load factors which would provide max effect for either bearing or sliding / eccentricity similar to AASHTO Figures C11.5.5-1 and C11.5.5.2.
10. Bouyancy has been included in sliding load combinations. A load factor of 0.0 has been used for bearing pressure load combinations since it is conservative to ignore sliding for these computations.

Strength	LC1	LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): gp max*(DCsub)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Strength	LC2	LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Bearing	LC3	LC3 - STRENGTH I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Sliding	LC4	LC4 - STRENGTH I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Bearing	LC5	LC5 - STRENGTH III BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Sliding	LC6	LC6 - STRENGTH III SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Bearing	LC7	LC7 - STRENGTH V BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Sliding	LC8	LC8 - STRENGTH V SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Extreme Bearing	LC9	LC9 - EXTREME EVENT I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+gEQ MAX*(LL+IM+PL+BR+LS)+1.0*(EQ)
Extreme Sliding	LC10	LC10 - EXTREME EVENT I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+gEQ MIN*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)
Extreme Bearing	LC11	LC11 - EXTREME EVENT II BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(CT)
Extreme Sliding	LC12	LC12 - EXTREME EVENT II SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(CT)

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

LRFD Load Combinations

Load Combinations : 3.1

↓ **N/A, Valid for Pile Design Only** ↓

NA (for Bottom row of piles) From Pile Design = **0**

Bottom Row to Edge of Toe = **0**

↓ **N/A, Valid for Pile Design Only** ↓

Distance of Pile Group N.A. From Footing Toe (See Pile Design Spreadsheet): **0.00 ft**

LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): $\gamma_{p,max}$*(DCsub)+$\gamma_{p,max}$*(EH)+$\gamma_{p,max}$*(EV)+$\gamma_{p,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC_{SUB}	1.25	31.52		289.21	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
ES	1.50	2.52		37.75	2.49		25.81
SUM		**54.86**		**617.42**	**2.49**		**25.81**

Distance of Vertical Force (V) From The Footing Toe	Offset of Pile Group N.A. From Original Location of V	Equivalent Moment Due to Offset of Pile Group N.A. From Original Location of V	**Mom. to Be Used On Pile Group = O.T. Mom. - Equivalent Mom.**	**Vertical Force to Be Used On Pile Group**	**Horizontal Force to Be Used On Pile Group**
11.25 ft	11.25 ft	617.4 k.ft	**-591.6 k.ft**	**54.9 kip**	**2.5 kip**

LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): $\gamma_{p,max}$*(DC+DW)+$\gamma_{p,max}$*(EH)+$\gamma_{p,max}$*(EV)+$\gamma_{p,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	58.50		536.71	0.00		0.00
DW	1.5	1.54		14.14	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
ES	1.50	2.52		37.75	2.49		25.81
SUM		**83.38**		**879.07**	**2.49**		**25.81**

↓ **N/A, Valid for Pile Design Only** ↓

10.54 ft	10.54 ft	879.1 k.ft	**-853.3 k.ft**	**83.4 kip**	**2.5 kip**

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General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.2

LC3 - STRENGTH I BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	58.50		536.71	0.00		0.00
DW	1.5	1.54		14.14	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
LL+IM+PL+BR+LS	1.75	17.27		158.47	1.49		30.88
WA	1.00	-24.02		-298.71	0.00		0.00
TU	0.50	0.00		0.000	0.0000		0.000
SUM		**74.11**		**701.08**	**1.49**		**30.88**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

9.46 ft	9.46 ft	701.1 k.ft	**-670.2 k.ft**	**74.1 kip**	**1.5 kip**

LC4 - STRENGTH I SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	42.12		386.43	0.00		0.00
DW	0.65	0.67		6.13	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.00	15.24		211.85	0.00		0.00
LL+IM+PL+BR+LS	1.75	17.27		158.47	1.49		30.88
WA (static)	1.00	-24.02		-298.71	0.00		0.00
TU	0.50	0.00		0.00	0.000		0.000
SUM		**51.52**		**468.63**	**1.49**		**30.88**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

9.10 ft	9.10 ft	468.6 k.ft	**-437.8 k.ft**	**51.5 kip**	**1.5 kip**

LC5 - STRENGTH III BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	58.50		536.71	0.00		0.00
DW	1.5	1.54		14.14	0.00		0.00
EH	1.425	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
WA (static)	1.00	-24.02		-298.71	0.00		0.00
WS	1.40	0.00		0.00	0.96		19.92
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**56.84**		**542.61**	**0.96**		**19.92**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

9.55 ft	9.55 ft	542.6 k.ft	**-522.7 k.ft**	**56.8 kip**	**1.0 kip**

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.3

LC6 - STRENGTH III SLIDING: $g_{p,min}$*(DC+DW)+$g_{p,max}$*(EH)+$g_{p,min}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.90	42.12		386.43	0.00		0.00
DW	0.65	0.67		6.13	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.00	15.24		211.85	0.00		0.00
WA	1.00	-24.02		-298.71	0.00		0.00
WS	1.40	0.00		0.00	0.96		19.92
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**34.25**		**310.17**	**0.96**		**19.92**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.06 ft	9.06 ft	310.2 k.ft	**-290.3 k.ft**	**34.2 kip**	**1.0 kip**

LC7 - STRENGTH V BEARING: $g_{p,max}$*(DC+DW)+$g_{p,max}$*(EH)+$g_{p,max}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	58.50		536.71	0.00		0.00
DW	1.5	1.54		14.14	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
LL+IM+PL+BR+LS	1.35	13.32		122.24	1.15		23.82
WA	1.00	-24.02		-298.71	0.00		0.00
WS	0.40	0.00		0.00	0.27		5.69
WL	1.00	0.00		0.00	0.14		2.95
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**70.16**		**664.86**	**1.57**		**32.46**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.48 ft	9.48 ft	664.9 k.ft	**-632.4 k.ft**	**70.2 kip**	**1.6 kip**

LC8 - STRENGTH V SLIDING: $g_{p,min}$*(DC+DW)+$g_{p,max}$*(EH)+$g_{p,min}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	42.12		386.43	0.00		0.00
DW	0.65	0.67		6.13	0.00		0.00
EH	1.425	0.24		4.47	0.00		0.00
EV	1	15.24		211.85	0.00		0.00
LL+IM+PL+BR+LS	1.35	13.32		122.24	1.15		23.82
WA	1.00	-24.02		-298.71	0.00		0.00
WS	0.40	0.00		0.00	0.27		5.69
WL	1.00	0.00		0.00	0.14		2.95
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**47.57**		**432.41**	**1.57**		**32.46**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.09 ft	9.09 ft	432.4 k.ft	**-400.0 k.ft**	**47.6 kip**	**1.6 kip**

PIER DESIGN - LOAD COMBINATIONS

TE

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC9 - EXTREME EVENT I BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$g_{EQ\,MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	58.50		536.71	0.00		0.00
DW	1.5	1.54		14.14	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.93		45.28	0.43		8.82
WA	0.00	0.00		0.00	0.00		0.00
EQ	1.00	7.66		140.49	8.31		136.38
SUM		**93.45**		**1027.09**	**8.74**		**145.20**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

10.99 ft	10.99 ft	1027.1 k.ft	**-881.9 k.ft**	**93.4 kip**	**8.7 kip**

LC10 - EXTREME EVENT I SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+$g_{EQ\,MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	42.12		386.43	0.00		0.00
DW	0.65	0.67		6.13	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.00	15.24		211.85	0.00		0.00
LL+IM+PL+BR+LS	0.00	0.00		0.00	0.00		0.00
WA (seismic)	1.00	-24.02		-298.71	0.00		0.00
EQ	1.00	7.66		140.49	8.31		136.38
SUM		**41.90**		**450.66**	**8.31**		**136.38**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

10.75 ft	10.75 ft	450.7 k.ft	**-314.3 k.ft**	**41.9 kip**	**8.3 kip**

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PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC11 - EXTREME EVENT II BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$g_{EQ\,MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	58.50		536.71	0.00		0.00
DW	1.5	1.54		14.14	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.35	20.58		285.99	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.93		0.00	0.43		8.82
WA	0.00	0.00		0.00	0.00		0.00
CT	1.00	9.87		0.00	0.00		0.00
SUM		**95.66**		**841.32**	**0.43**		**8.82**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

8.79 ft	8.79 ft	841.3 k.ft	**-832.5 k.ft**	**95.7 kip**	**0.4 kip**

LC12 - EXTREME EVENT II SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+$g_{EQ\,MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	42.12		386.43	0.00		0.00
DW	0.65	0.67		6.13	0.00		0.00
EH	1.43	0.24		4.47	0.00		0.00
EV	1.00	15.24		211.85	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.93		0.00	0.00		0.00
WA (seismic)	1.00	-24.02		-298.71	0.00		0.00
CT	1.00	9.87		0.00	0.00		0.00
SUM		**49.05**		**310.17**	**0.00**		**0.00**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

6.32 ft	6.32 ft	310.2 k.ft	**-310.2 k.ft**	**49.1 kip**	**0.0 kip**

PIER DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 9, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Check Bearing Resistance (per AASHTO 11.6.3.2) -- ON SOIL

Stability : 1.0

If supported on soil, the vertical stress (σ_v) shall be calculated assuming a uniformly distributed pressure (V) over an effective base area (B-2e). AASHTO Fig 11.6.3.2-1 —> $qr\ /\ \Phi\beta = q_n =$

If supported on rock, the vertical stress (σ_v) shall be calculated assuming a linearly distributed pressure over an effective base area. AASHTO Fig 11.6.3.2-2 —> $qr\ /\ \Phi\beta = q_n =$

Factored Bearing Resistance, qr:	$qr = \Phi\beta * q_n =$ 7.43 ksf	<--- Note per Geotech, this is factored net bearing resistance		
Strength Bearing Resistance Factor, $\Phi\beta$ (AASHTO Table 11.5.7-1):	0.45	$qr = \Phi\beta * q_n =$ 7.43 ksf —> $qr\ /\ \Phi\beta = qn =$	16.50 ksf	Note —> See AASHTO Table 11.5.7-1 to determine $\Phi\beta$ Factor
Extreme Event Bearing Resistance Factor, $\Phi\beta$ (AASHTO 10.5.5.3.3):	1.00	$qr = \Phi\beta * q_n =$ 7.43 ksf —> $qr\ /\ \Phi\beta = qn =$	7.43 ksf	

	LOAD COMBINATION	Vertical Force (Kips)	Resisting Moment (Ft x K)	Overturn Moment (Ft x K)	Mnet (Ft x K)	Eccentricty from Toe, et=Mnet/V (Ft)	Eccentricty from CL, e=B/2-et (Ft)	σ_v on soil (ksf)	$\sigma_{v\,max}$ on rock (ksf)	$\sigma_{v\,min}$ on rock (ksf)	$\sigma_v < \Phi\beta * q_n$	
Strength	LC1	54.86	617.42	25.81	591.61	10.78	-1.61	2.54	1.42	4.56	OK	
Strength	LC2	83.38	879.07	25.81	853.26	10.23	-1.06	4.07	2.97	6.12	OK	
Bearing	LC3	74.11	701.08	30.88	670.20	9.04	0.13	4.10	4.21	3.87	OK	
Sliding	LC4	51.52	468.63	30.88	437.76	8.50	0.68	3.03	3.43	2.19	OK	<--*N/A Sliding Combination
Bearing	LC5	56.84	542.61	19.92	522.69	9.20	-0.02	3.09	3.08	3.12	OK	
Sliding	LC6	34.25	310.17	19.92	290.25	8.48	0.70	2.02	2.29	1.44	OK	<--*N/A Sliding Combination
Bearing	LC7	70.16	664.86	32.46	632.40	9.01	0.16	3.89	4.02	3.62	OK	
Sliding	LC8	47.57	432.41	32.46	399.96	8.41	0.77	2.83	3.24	1.94	OK	<--*N/A Sliding Combination
Ex. Bearing	LC9	93.45	1027.09	145.20	881.88	9.44	-0.26	4.95	4.66	5.53	OK	
Ex. Sliding	LC10	41.90	450.66	136.38	314.28	7.50	1.68	2.79	3.53	1.03	OK	<--*N/A Ex. Sliding Combination
Ex. Bearing	LC11	95.66	841.32	8.82	832.49	8.70	0.47	5.50	6.02	4.41	OK	
Ex. Sliding	LC12	49.05	310.17	0.00	310.17	6.32	2.85	3.88	5.17	0.18	OK	<--*N/A Ex. Sliding Combination

* Sliding Load Combinations are Not Applicable for checking the Bearing

PIER DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Check Overturning (per AASHTO 11.6.3.3) -- ON SOIL

Stability : 2.0

e allowable (ftgs on soil): 4.59 ft
e allowable (ftgs on rock): 6.88 ft
If e < e allowable, Overturning is OK:

	LOAD COMBINATION	Eccentricty from CL, e=B/2-et (Ft)	Check Overturning	
Strength	**LC1**	-1.61	OK	
Strength	**LC2**	-1.06	OK	
Bearing	**LC3**	0.13	OK	
Sliding	**LC4**	0.68	**OK**	**<--*N/A Sliding Combination**
Bearing	**LC5**	-0.02	OK	
Sliding	**LC6**	0.70	**OK**	**<--*N/A Sliding Combination**
Bearing	**LC7**	0.16	OK	
Sliding	**LC8**	0.77	**OK**	**<--*N/A Sliding Combination**
Ex. Bearing	**LC9**	-0.26	OK	
Ex. Sliding	**LC10**	1.68	**OK**	**<--*N/A Ex. Sliding Combination**
Ex. Bearing	**LC11**	0.47	OK	
Ex. Sliding	**LC12**	2.85	**OK**	**<--*N/A Ex. Sliding Combination**

*** Sliding Load Combinations are Not Applicable for checking Overturning**

PIER DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Check Sliding (per AASHTO 10.6.3.4)

Stability : 3.0

Ignore Passive Resistance of Soil per MassHighway

Strength Sliding Resistance Factor, Φτ (AASHTO Table 11.5.7-1):	1.00	
Extreme Event Sliding Resistance Factor, Φϖ (AASHTO 10.5.5.3.3):	1.00	
Internal Friction Angle of Drained Soil, Φ_f:	33.00	degrees
tan δ: = tan Φ_f (per AASHTO 10.6.3.4-2):	0.65	for concrete against soil. Multiply by 0.8 for precast concrete footing

	LOAD COMBINATION	Vertical Force (Kips)	Rt = V * tan δ: (Kips)	Φτ (Strength) Φω (Extreme) (Kips)	Nom. Sliding Resistance Φτ*Rt (Kips)	Horiz Force (Kips)	Check Sliding	
Strength	LC1	54.86	35.63	1.00	35.63	2.49	OK	<--*N/A Strength Combination
Strength	LC2	83.38	54.15	1.00	54.15	2.49	OK	<--*N/A Strength Combination
Bearing	LC3	74.11	48.13	1.00	48.13	1.49	OK	<--*N/A Bearing Combination
Sliding	LC4	51.52	33.46	1.00	33.46	1.49	OK	
Bearing	LC5	56.84	36.91	1.00	36.91	0.96	OK	<--*N/A Bearing Combination
Sliding	LC6	34.25	22.24	1.00	22.24	0.96	OK	
Bearing	LC7	70.16	45.56	1.00	45.56	1.57	OK	<--*N/A Bearing Combination
Sliding	LC8	47.57	30.89	1.00	30.89	1.57	OK	
Ex. Bearing	LC9	93.45	60.69	1.00	60.69	8.74	OK	<--*N/A Ex. Bearing Combination
Ex. Sliding	LC10	41.90	27.21	1.00	27.21	8.31	OK	
Ex. Bearing	LC11	95.66	62.12	0.65	40.34	0.00	OK	<--*N/A Ex. Bearing Combination
Ex. Sliding	LC12	49.05	31.85	0.65	20.69	0.00	OK	

Results Summary:

Stability : 4.0

STABILITY RESULTS:

LOAD COMBINATION:	BEARING RESISTANCE	OVERTURNING	SLIDING	
LC1	OK	OK	OK	<== Construction
LC2	OK	OK	OK	<== Construction
LC3	OK	OK	OK	
LC4	OK	OK	OK	
LC5	OK	OK	OK	
LC6	OK	OK	OK	
LC7	OK	OK	OK	
LC8	OK	OK	OK	
LC9	OK	OK	OK	
LC10	OK	OK	OK	
LC11	OK	OK	OK	
LC12	OK	OK	OK	

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PIER DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).
Pier Options
Khost Bridge Notes

Design Parameters

Reinforcement : 1.0

GEOMETRY

H of Footing, h :	4.92	ft
bw (per linear ft of wall) :	12.00	in

MATERIAL PROPERTIES

Compressive Strength: f'c :	4.00	ksi	
Min Yield Strength: fy :	60.00	ksi	
Max. Agg. Size :	1.50	in	
Es :	29000	ksi	AASHTO 5.4.3.2
Tension Reinforcement Strain: ε_s :	0.002	ε_s = fy / Es	
β :	1.881		AASHTO EQ 5.8.3.4.2-1

Design Heel and Toe Reinforcement

Reinforcement : 2.1

FACTORED HEEL DESIGN LOADS	Load Factor, γ_p AASHTO Table 3.4.1-2	Vertical Force & Design Shear (Kips)	Arm (Feet)	Design Moment (Ft x K)
DC (Heel Concrete)	1.25	6.19	3.36	20.80
EV (Heel Soil)	1.35	18.6429332	3.36	62.59
EH (Vertical Component)	1.43	0.48	6.72	3.21
LS	1.75	0.00	3.36	0.00
SUM		**25.32**		**86.60**

* See load combs, Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2) for the above Load Factors

* EH (Vertical Component) <-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.

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Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.2

Footing Toe Width, BF: 6.72 ft

FACTORED TOE DESIGN LOADS LOAD COMBINATION	σ_v Factored Toe Pressure (ksf)	Factored Toe Shear (Kips)	Factored Toe Moment (Ft x K)
LC1	2.54	17.08	57.34
LC2	4.07	27.35	91.84
LC3	4.10	27.51	92.37
LC4	3.03	20.36	68.35
LC5	3.09	20.75	69.67
LC6	2.02	13.57	45.55
LC7	3.89	26.13	87.74
LC8	2.83	19.00	63.78
LC9	4.95	33.25	111.63
LC10	2.79	18.76	62.98
MAX		**33.25**	**111.63**

Note: Based on AASHTO 10.6.5, the structural design of an eccentrically loaded foundation can assume a triangular or eccentrically loaded area. This spreadsheet conservatively assumes a uniform pressure of sv max over the toe of the footing. Based on AASHTO Figure C5.13.3.6.1-1, The toe shear can be computed at a distance dv from the face of support. This spreadsheet computes it at the support, which is conservative.

10.6.5—Structural Design

The structural design of footings shall comply with the requirements given in Section 5.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions.

FOOTING HEEL REINF (TOP BARS):

USE #	7.00	@	6.00 IN
Abar =	0.60	in^2	
dbar =	0.88	in	
Asprov =	1.20	in^2	

FOOTING TOE REINF (BOTTOM BARS):

USE #	7.00	@	6.00 IN
Abar =	0.60	in^2	
dbar =	0.88	in	
Asprov =	1.20	in^2	

CRITICAL SECTION FOR WALLS

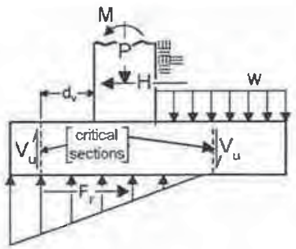

Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

CRITICAL SECTION FOR ABUTMENTS

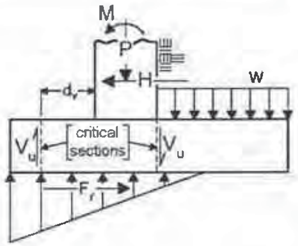

Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

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Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.3

CHECK FLEXURAL RESISTANCE	HEEL	TOE	AASHTO 5.7, 5.7.2.2, 5.7.3.2, 5.7.3.2.2	
Factored Moment, Mu =	86.60	111.63	k*ft	
Resistance Factor, phi: Φ =	0.90	0.90		AASHTO 5.5.4.2
Assume Cover, dc =	2.00	3.00	in	ACI 318-08 - 7.7
Shear Depth: ds =	56.60	55.60	in = h - cover - 1/2db(main)	
Depth of Equivalent Stress Block: a =	1.76	1.76	in = c*β1 = Asfy/0.85f'cb	AASHTO Eq. 5.7.3.1.1-4
Nominal Flexural resistance, Mn =	334.32	328.32	kip ft = [Asfy(ds-a/2)]/12	AASHTO Eq. 5.7.3.2.2-1
Factored Resistance, ΦMn =	300.89	295.49		AASHTO Eq. 5.7.3.2.1-1
As required for Mu:	0.35	0.45	in^2	
Flexure OK?	**OK**	**OK**		

CHECK MINIMUM REINFORCEMENT	HEEL	TOE	AASHTO 5.7.3.3.2	
Section Modulud: Sc =	6971.44	6971.44	in^3	
Compressive Strength: f'c =	4.00	4.00	ksi	
Modulus of Rupture: fr =	0.74	0.74	ksi = $0.37*(f'c)^{1/2}$	AASHTO 5.4.2.6
Cracking Moment: Mcr = Sc*fr =	429.91	429.91	kip ft	AASHTO Eq. 5.7.3.2.2-1
Factored Flexural Resistance: Mr1 = 1.2*Mcr =	515.89	515.89	kip ft	
Factored Moment, Mu =	86.60	111.63	k*ft	
Factored Flexural Resistance: Mr2 = 1.33*Mu =	115.18	148.46	kip ft	
Controlling Mr = min(Mr1, Mr2)	115.18	148.46	kip ft	
Factored Resistance, phi*Mn =	300.89	295.49		AASHTO Eq. 5.7.3.2.1-1
As required for Mr:	0.4549	0.5981	in^2	
As required for Temp Steel (#4@18"):	0.1333	0.1333	in^2	
As provided =	1.20	1.20	in^2	
Min Reinforcement OK?	**OK**	**OK**		

CHECK CRACK CONTROL BY DIST REINF.	HEEL	TOE	AASHTO 5.7.3.4, 5.10.3.1		
Exposure Factor: γ_e =	0.75	0.75	Class 2 Exposure		ASSHTO 5.7.3.4
β_s factor=	1.05	1.08		β_s factor = 1 + (dc / 0.7* (h-dc))	ASSHTO 5.7.3.4-1
f_{ss} =	36	36	ksi	f_{ss} = .6*fy	
s_{max} =	8.89	8.55	in	s_{max} < = 700 ge / βs fss	ASSHTO 5.7.3.4-1
s_{min} =	3.13	3.13	in	s_{min} = max(1.5*db,1.5*agg,1.5")+db	AASHTO 5.10.3.1.1
SPACING OK?	**OK**	**OK**			

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Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.4

CHECK SHEAR RESISTANCE	HEEL	TOE	AASHTO 5.13.3.6, 5.8.3		
Factored Shear Force, Vu =	25.32	33.25	kips		
Factored Moment, Mu =	86.60	111.63	k*in		
Es =	29000	29000			AASHTO 5.4.3.2
Resistance Factor, phi: Φ =	0.90	0.90			AASHTO 5.5.4.2
bw (per linear ft of wall) =	12.00	12.00	in		
Effective Depth: dv =	55.72	54.72	in	dv = max((ds-a/2),max(0.9ds, 0.72h))	AASHTO 5.8.2.9
H of Ftg, h:	59.04	59.04	in		
bw (per linear ft of ftg) =	12.00	12.00	in		
Area of Conc on Tension Side, Ac =	354.24	354.24	in	Ac = h*bw/2 =	
As (flexural) provd =	1.20	1.20	in^2		
Max. Size of Coarse Aggregate, ag =	1.50	1.50	in		
Mu min =	1410.57	1819.29	k*in	Mu min = Vu*dv =	
Mu (controlling) =	1410.57	1819.29	k*in		
Spg between top and bottom reinf, sx =	54.04	54.04	in		
Crack spg parameter, sxe =	35.01	35.01		sxe = 1.38*sx/(ag+0.63)	
Strain = ε_s=	0.0015	0.0019		ε_s=(Mu/dv+Vu)/(Es*As)	AASHTO EQ 5.8.3.4.2-4
Θ =	147.67	193.94		Θ = 29+3500*ε_s	AASHTO EQ 5.8.3.4.2-3
β =	1.58	1.36		β = 4.8/(1+750ε_s)*(51/(39+s_{xe}))	
Nom Shear Resistance, Vn1 =	668.64	656.64	kips	Vn1 = 0.25*f'c*bv*dv	AASHTO 5.8.3.3-2
Nominal Shear Resistance: Vn2 = Vc =	66.84	56.42	kips	Vn2 = Vc = 0.0316*β*f'c.5*bv*dv	AASHTO 5.8.3.3-3
Nom Shear Resistance, Vn =	66.84	56.42	kips	Vn = min (Vn1, Vn2)	
phi*Vn =	60.16	50.77			
Shear OK?	**OK**	**OK**			
Opposite Face Reinf As provd. =	1.20	1.20	in^2		
As min crack =	1.95	1.95	in^2	As min crack = 0.003*b*sx	
min (As front, back) > As min ?	**N/A**	**N/A**			

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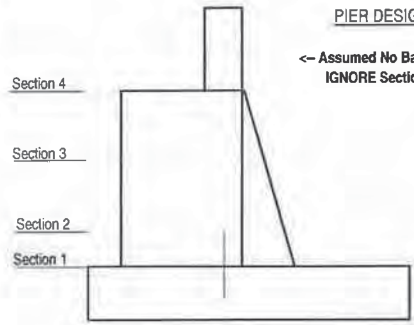
Stem Reinforcement

Reinforcement : 3.0

1. Reinforcement does not need to be checked for construction loading since that is a temporary load case.
Check the stem reinforcement at various locations along the stem and at the base of the backwall.

Height of Stem plus Backwall, h = H - F =	15.82	ft
Height of Backwall =	0.00	ft
Ftg Dowel Lap Length:	7.00	ft
Width of Stem at the Base:	4.92	ft
Width of Backwall:	0.00	ft
Width of Batter:	0.00	ft

Section	Height of h	Height from top	Width Batter	Width conc	
1	1.00	15.82	0.00	4.92	<=== This section is at the bottom of the stem.
2	0.56	8.82	0.00	4.92	<=== This section is at the top of the footing dowel.
3	0.28	4.41	0.00	4.92	<=== This section is halfway in between top of footing dowel and top of batter.
4	0.00	0.00		0.00	<=== This section is at the base of the backwall



Horizontal Earth Pressure, EH at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	20.73944				0.00	4.37	0.00
Top of Ftg	15.81944				0.00	5.27	0.00
Top of Dowel	8.81944				0.00	2.94	0.00
Mid-Height	4.40972				0.00	1.47	0.00
Bot of Backwall	0				0.00	0.00	0.00

Live Load Surcharge, LS at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	20.73944				0.00	10.37	0.00
Top of Ftg	15.81944				0.00	7.91	0.00
Top of Dowel	8.81944				0.00	4.41	0.00
Mid-Height	4.40972				0.00	2.20	0.00
Bot of Backwall	0				0.00	0.00	0.00

Seismic Load, EQ at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	20.73944				8.31		136.38
Top of Ftg	15.81944				6.34	7.91	50.15
Top of Dowel	8.81944				3.53	4.41	15.59
Mid-Height	4.40972				1.77	2.20	3.90
Bot of Backwall	0				0.00	0.00	0.00

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Stem Reinforcement Cont.

Reinforcement : 3.1

Load Combination - STRENGTH I		At Top of Ftg		Top of Dowel		Mid-Height Abut		Bot of Backwall	
LOAD	Load Factor	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)
EH	1.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LS	1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SUM		**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Load Combination - EXTREME EVENT I		At Top of Ftg		Top of Dowel		Mid-Height Abut		Bot of Backwall	
LOAD	Load Factor	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)
EH	1.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LS	0.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EQ	1.00	6.34	50.15	3.53	15.59	1.77	3.90	0.00	0.00
SUM		**6.34**	**50.15**	**3.53**	**15.59**	**1.77**	**3.90**	**0.00**	**0.00**

CHECK FLEXURAL RESISTANCE	**SECT 1**	**SECT 2**	**SECT 3**	**SECT 4**	**AASHTO 5.7**		
Section Height / Location =	15.82	8.82	4.41	0.00	ft		**<-- Assumed No Backwall - IGNORE Section 4**
Factored Moment, Mu =	50.15	15.59	3.90	0.00	k*ft		
Resistance Factor, phi: Φ =	0.90	0.90	0.90	0.90		AASHTO 5.5.4.2	
[redacted]	4.92	4.92	4.92	0.00	ft		
Cover, dc =	2.00	2.00	2.00	2.00	in	ACI 318-08: Sec 7.7.1	<-- 2" for Concrete exposed to earth or weather: No. 6 thru No 18 bars
BAR # =	7.00	7.00	7.00	0.00			
SPACING =	8.00	8.00	8.00	8.00	in		
Main Abar =	0.60	0.60	0.60	0.00	in^2		
Main db =	0.875	0.875	0.875	0.000	in		
As provd. =	0.90	0.90	0.90	0.00	in^2		
Shear Depth: ds =	56.60	56.60	56.60	-2.00	in. = h - cover - 1/2db(main)		
Depth of Equivalent Stress Block: a =	1.32	1.32	1.32	0.00	in = c*β1 = Asfy/0.85f'cb	AASHTO 5.7.2.2, 5.7.3.2	
Nominal Flexural resistance, Mn =	251.73	251.73	251.73	0.00	kip ft = [Asfy(ds-a/2)]/12	AASHTO 5.7.3.2.2	
Factored Resistance, phi*Mn =	226.56	226.56	226.56	0.00		AASHTO Eq. 5.7.3.2.1-1	
As required for Mu:	0.1997	0.0620	0.0155	0.0000	in^2		
Flexure OK?	**OK**	**OK**	**OK**	**N/A**			

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Stem Reinforcement Cont.

Reinforcement : 3.2

<-- Assumed No Backwall - IGNORE Section 4

CHECK MINIMUM REINFORCEMENT	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.7.3.3.2	
Section Modulud: Sc =	6971.44	6971.44	6971.44	0.00	in^3	
Modulus of Rupture: fr =	0.74	0.74	0.74	0.74	ksi = $0.37*(f'c)^{1/2}$	AASHTO 5.4.2.6
Cracking Moment: Mcr = Sc*fr =	429.91	429.91	429.91	0.00	kip ft	
Factored Flexural Resistance: Mr1 = 1.2*Mcr =	515.89	515.89	515.89	0.00	kip ft	
Factored Moment, Mu =	50.15	15.59	3.90	0.00	k*ft	
Factored Flexural Resistance: Mr2 = 1.33*Mu =	66.70	20.73	5.18	0.00	kip ft	
Controlling Mr = min(Mr1, Mr2)	66.70	20.73	5.18	0.00	kip ft	
Factored Resistance, phi*Mn =	226.56	226.56	226.56	0.00		AASHTO Eq. 5.7.3.2.1-1
As required for Mr:	0.2628	0.0815	0.0204	-2.7200	in^2	
As provided =	0.90	0.90	0.90	0.00	in^2	
Min Reinforcement OK?	**OK**	**OK**	**OK**	**N/A**		

<-- Assumed No Backwall - IGNORE Section 4

CHECK CRACK CONTROL BY DIST REINF.	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.7.3.4, 5.10.3.1	
Exposure Factor: γ_e =	0.75	0.75	0.75	0.75	Class 2 Exposure	ASSHTO 5.7.3.4
H of Stem, h:	59.04	59.04	59.04	0.00	in	
β_s factor=	1.05	1.05	1.05	-0.43	1 + (dc / 0.7* (h-dc))	ASSHTO 5.7.3.4-1
f_{ss} =	36	36.00	36.00	36.00	ksi = .6*fy	
s_{max} =	9.89	9.89	9.89	-38.03	in < = 700 γ_e / β_s f_{ss}	ASSHTO 5.7.3.4-1
Main db =	0.875	0.875	0.875	0.000	in	
s_{min} = max(1.5*db,1.5*agg,1.5")+db =	3.13	3.13	3.13	2.25	in	ASSHTO 5.10.3.1.1
SPACING =	8.00	8.00	8.00	8.00	in	
SPACING OK?	**OK**	**OK**	**OK**	**N/A**		

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Stem Reinforcement Cont.

Reinforcement : 3.3

CHECK SHEAR TRANSFER	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.8.4.1, 5.8.4.3, 5.8.4.4
Cohesion Factor, c =	0.075	0.075	0.075	0.075	ksi, assumes CJ not intentionally roughened
Friction Factor, μ =	0.6	0.60	0.60	0.60	
Fraction of strength for interface shear, K1 =	0.2	0.20	0.20	0.20	
Limiting Interface Shear Resistance, K2 =	0.8	0.80	0.80	0.80	ksi
Lvi = H of Stem, h:	59.04	59.04	59.04	0.00	ft
bvi = bw (per linear ft of wall) =	12.00	12.00	12.00	12.00	in
Interface Area, Acv = Lvi*bvi =	708.48	708.48	708.48	0.00	in^2
Back Face (Flexural) As provd. =	0.90	0.90	0.90	0.00	in^2
Front Face (Dowels) As provd. =	1.58	1.58	1.58	1.58	in^2
Interface Reinf Provided, Avf = As back+front =	2.48	2.48	2.48	1.58	in^2
Vni = c*Acv+μ*Avf*Fy =	142.42	142.42	142.42	56.88	kips
Vni max1 = K1*f'c*Acv =	566.78	566.78	566.78	0.00	kips
Vni max2 = K2*Acv =	566.78	566.78	566.78	0.00	kips
Vni (controlling) =	142.42	142.42	142.42	0.00	kips
Fact. Interface Shear Resistance, Vri=phiVni =	128.17	128.17	128.17	0.00	kips
Fact. Interface Shear Load, Vui = Vu =	6.34	3.53	1.77	0.00	kips
Vu < Vri ?	**OK**	**OK**	**OK**	**N/A**	
Min Interface Shear Reinf, Avf = 0.05*Acv/Fy =	0.590	0.590	0.590	0.000	in^2
Avf > Avfmin ?	**OK**	**OK**	**OK**	**N/A**	

<-- Assumed No Backwall - IGNORE Section 4

5.8.4.3—Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

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Stem Reinforcement Cont.

Reinforcement : 3.4

CHECK SHEAR RESISTANCE	**SECT 1**	**SECT 2**	**SECT 3**	**SECT 4**	**AASHTO 5.8.2, 5.8.3.3, 5.8.3.4.2**
Factored Shear Force, Vu =	6.34	3.53	1.77	0.00	kips
Factored Moment, Mu =	601.79	0.00	0.00	0.00	k*in
Resistance Factor, phi: Φ =	0.90	0.90	0.90	0.90	AASHTO 5.5.4.2
Effective Depth: dv =	55.94	55.94	55.94	0.00	in=max((ds-a/2),max(0.9ds,0.72h)) AASHTO 5.8.2.9
H of Stem, h:	59.04	59.04	59.04	0.00	in
Area of Conc on Tension Side, Ac = h*bw/2 =	354.24	354.24	354.24	0.00	in
As (flexural, back face) provd =	0.90	0.90	0.90	0.00	in^2
Max. Size of Coarse Aggregate, ag =	1.50	1.50	1.50	1.50	in
Mu min = Vu*dv =	354.67	197.73	98.87	0.00	k*in
Mu (controlling) =	601.79	197.73	98.87	0.00	k*in
sx = dv	55.94	55.94	55.94	0.00	in ---> See Figure 5.8.3.4.2-3 (Case a)
Crack spg parameter, sxe = 1.38*sx/(ag+0.63) =	36.24	36.24	36.24	0.00	
Strain = ε_s=(Mu/d+Vu)/(Es*As) =	0.0007	0.0003	0.0001	#DIV/0!	
Θ = 29+35000*ε_s =	66.49	27.49	13.75	#DIV/0!	
β = 4.8/(1+750εs)*(51/(39+s_{xe}) =	2.18	2.70	2.95	#DIV/0!	
Nom Shear Resistance, Vn1 =	671.29	671.29	671.29	0.00	kips, Vn = 0.25*f'c*bv*dv AASHTO 5.8.3.3-2
Nominal Shear Resistance: Vn2 = Vc =	92.56	114.72	125.30	#DIV/0!	kips, 0.0316*β*f'c$^{.5}$*bv*dv AASHTO 5.8.3.3-3
Nom Shear Resistance, Vn = min (Vn1, Vn2) =	92.56	114.72	125.30	#DIV/0!	kips
phi*Vn =	83.30	103.25	112.77	#DIV/0!	
Shear OK?	**OK**	**OK**	**OK**	**N/A**	
Front Face (Dowels) As provd. =	1.58	1.58	1.58	1.58	in^2
As min crack = 0.003*b*sx =	2.01	2.01	2.01	0.00	in^2 ---> Only Applicable for Figure 5.8.3.4.2-3 Case B
min (As front, back) > As min ?	**N/A**	**N/A**	**N/A**	**N/A**	

<-- Assumed No Backwall - IGNORE Section 4

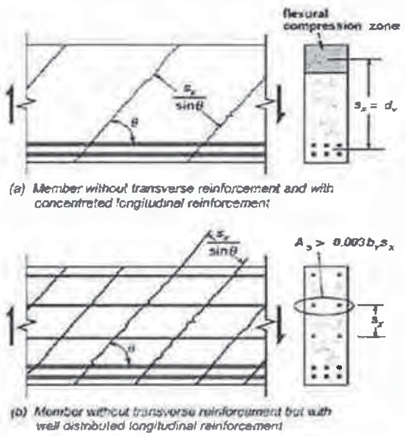

Figure 5.8.3.4.2-3—Definition of Crack Spacing Parameter, s_x

Crack Spacing Parameter, Sx --> Case = Case A

PIER DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier Design	**Date:**	October 3, 2014

Results Summary:

Reinforcement : 4.0

REINFORCEMENT RESULTS:

REINF. RATIO = As Provided / As Required

		STIRUP #	BAR #	SPAC.	REINF. RATIO	FLEX OK?	LBS./ L.F.	LENGTH OF BAR	No. Bars per ft	Wt. of bar PER L.F.	As/LF	SHEAR OK?	
A	TOE(bot):	---	7.00	6.00	2.01	OK	145.78	17.85	2.00	4.08	1.20	OK	
B	HEEL(top):	---	7.00	6.00	2.64	OK	145.78	17.85	2.00	4.08	1.20	OK	
C	STEM 1 (at top of ftg):	0.00	7.00	8.00	3.43	OK	32.16	7.00	1.50	3.06	0.90	OK	
D	STEM 2 (at top of ftg dwl - backface):	0.00	7.00	8.00	11.05	OK	36.93	8.04	1.50	3.06	0.90	OK	
E	STEM 3 (midpt back face):	0.00	7.00	8.00	44.22	OK	18.47	4.02	1.50	3.06	0.90	OK	
F	STEM 4 (at bot of bw):	0.00	0.00	8.00	0.00	N/A	0.00	8.04	1.50	0.00	0.00	N/A	<— N/A No Backwall
G	STEM 5 (front face):	0.00	5.00	12.00	---	---	8.48	8.04	1.00	1.05	0.31		
H	STEM 6 (front face dowels):	0.00	8.00	6.00	---	---	21.51	2.00	2.00	5.38	1.58		
I	FOOTING (TOP):	0.00	6.00	12.00	---	---	27.47	1.00	18.35	1.50	0.44		
J	FOOTING (BOT.)	0.00	6.00	12.00	---	---	27.47	1.00	18.35	1.50	0.44		
K	STEM (longitudinal):	0.00	4.00	12.00	---	---	21.53	1.00	31.64	0.68	0.20		
	TOTAL WT. STEEL/FT OF ABUT.=						485.57	LBS./LF					

<u>SUBSTRUCTURE DESIGN</u>

WALL DESIGN

* WALL STABILITY DESIGN

CANTILEVER WALL DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

References:

AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012	AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011
ACI 318-08 Building Code Requirements for Structural Concrete, 2005	

General Notes: **This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).**

Wall Design | **Khost Bridge Notes**

Project Notes:

General Design Parameters

Input Section : 1.0

GEOMETRY INFORMATION INPUT:

PROPOSED TOP OF ROADWAY ELEV:			ft		m
PROPOSED TOP OF BACKWALL ELEV:		5972.06	ft	1820.750	m
PROPOSED BRIDGE SEAT ELEV:	H_Backwall = 0.00 ft	5972.06	ft	1820.750	m
PROPOSED TOP OF FOOTING ELEV:	H_Footing = 3.28 ft	5948.28	ft	1813.500	m
PROPOSED BOT. OF FOOTING ELEV:		5945.00	ft	1812.500	m
ELEVATION OF HIGH WATER:	FOR NO WATER = 0.00	5966.88	ft	1819.170	m
PROPOSED BRIDGE SEAT WIDTH:		1.97	ft	0.601	m
PROPOSED BACKWALL WIDTH:		0.00	ft	0.000	m
ABUTMENT/WALL DESIGN LENGTH:	1.00 Design for:	1.00	ft	0.305	m
FOOTING LENGTH	Design for:	1.00	ft	0.305	m

DW CALCULATION INPUT:

WEARING SURFACE DEPTH:	0.00 IN x 1. Layers	0.00	ft <---	0.000	m
ROADWAY WIDTH:		0.00	ft	0.000	m
BRIDGE SPAN:	**Total Length = 0**	0.00	ft	0.000	m
NUMBER OF GIRDERS:		0			

MATERIAL PROPERTIES:

CUBIC WEIGHT CONCRETE:	150.00	pcf
COMP. STRENGTH OF CONC. = F'c:	4.00	ksi
MAXIMUM SIZE OF COARSE AGGREGATE	1.50	in
TENSILE STRENGTH OF REBAR = Fy:	60.00	ksi
CUBIC WEIGHT OF HOT MIX ASPHALT (HMA):	165.00	pcf

GEOTECHNICAL INFORMATION:

NOMINAL BEARING RESISTANCE (CAPACITY), qn:	17.78	ksf	<-- Per Geotech Report
FACTORED BEARING RESISTANCE, qr :	8.00	ksf	<-- Per Geotech Report
WEIGHT OF SOIL BACKFILL:	148.00	Lbs/CF	<-- Per Geotech Report
WALL ON ROCK?	N	(Y OR N)	
WALL ON PILES?	N	(Y OR N)	
GRAVITY WALL?	N	(Y OR N)	
BETA: SLOPE OF BACKFILL:	0.00	DEG	<-- Per Civil
THETA: BATTER ANGLE BACKWALL:	81.04	DEG	AASHTO Table 3.11.5.3-1
PHI: FRICTION ANGLE OF BACKFILL:	33.00	DEG	<-- Per Geotech Report
DELTA: ANGLE BACKWALL FRICTION:	22.00	DEG	<-- Assumed δ=2/3 (ϕ)

WALL DESIGN - N/A

-

CANTILEVER ABUTMENT DESIGN	N
GRAVITY ABUTMENT DESIGN	N
CANTILEVER WALL DESIGN	Y
GRAVITY WALL DESIGN	N
PIER DESIGN	N

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CANTILEVER WALL DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

General Loading Parameters

Input Section : 2.0

LIVE LOAD INFORMATION:

APPROACH SLAB:	N	(Y OR N) <-- Wall Design - No Approach Slab
ROADWAY WITHIN H/2 OF TOP OF WALL:	N	(Y OR N)
Live Load Surcharge to be Considered?:	N	
SURCHARGE HEIGHT:	0.00	ft REF: Table 3.11.6.4-2
Construction Surcharge, q:	250.00	psf REF: C3.4.2.1

SEISMIC LOAD INFORMATION:

WALL RESTRAINED HORZ. MOVMT.(Y/N):	N	(Y OR N)
SEISMIC ACCELERATION COEFF. A:	0.290	REF: FIG.3.10.2.1-2, AASHTO
SEISMIC CATEGORY:	D	<--Assumed based on Location & AASHTO Seimic Design Guide

RAILING CLASS: S3-TL4 (CT) (PER MASSDOT LRFD BRIDGE MANUAL PART 1) 3.3.2.2 **<--- N/A**

Horizontal Railing Design Load	0.00	kips
Horizontal Railing Impact Length	0.00	ft
Wall Height+Rail Height	0.00	ft
Distributed Horizontal Railing Design Load @ top of wall	0.00	klf
Distributed Horizontal Railing Design Load @ bottom of wall	0.00	klf/wall height
Railing Dead Load	0.00	
Additional Moment From Railing Impact	0.00	<--- **Note:** The added moment from top of railing to bottom of railing is distributed along bottom of footing*

STREAM PRESSURE

Pmax	**0.00**	psf
Consider Stream Flow:	**N**	<--- Do not include stream pressure for the wall.

SURCHARGE HEIGHT (Per ASSHTO 3.11.6.4 Live Load Surchage)

ABUTMENTS (N/A for PIERS) ---> Table 3.11.6.4-1

Table 3.11.6.4-1 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpindicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5	4
10	3
>20	2

Surcharge Height = 0.00 ft

RETAINING WALLS ---> Table 3.11.6.4-2

See Table 3.11.6.4-2 for Equivalent Height of Soil for Vechicular Loading on Retaining Walls Parallel to Traffic.

Retaining Wall Height (ft)	heq (ft) Distance from wall backface to edge of traffic.	
	0.0 ft	≥1.0 ft
5	5	2
10	3.5	2
>20	2	2

Distance from wall backface to edge of traffic = 0.0 ft

Surcharge Height = 0.00 ft

Note: **See 3.11.6.5 for Possible Reduction of Surcharge**

CANTILEVER WALL DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Superstructure Loading Parameters

Input Section : 3.0

ADDITIONAL LOADS ON STRUCTURE **WALL DESIGN - N/A**

(load is per linear foot of structure (Abutment/ Pier/ Wall) NOT the Footing, arm from front edge of bridge seat)

LOADS		LOAD (klf)	ARM (feet)				
(DC+DW), SUPERSTRUCT. DEAD LOAD:	DL	0.00	0.99	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	N	<-- N/A For Wall Design
DC (Structural Components & nonstructural attachments)	DC	0.00	0.99	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	N	<-- N/A For Wall Design
DW (Wearing Surface & Utilities)	DW	0.00	0.99	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	N	<-- N/A For Wall Design
(LL+IM+PL), LIVE LOAD, IMPACT AND PED LL:	LL+IM+PL	0.00	0.99	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	N	<-- N/A For Wall Design
WS, WIND LOAD ON STRUCTURE:	WS	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	N	<-- N/A For Wall Design
WL, WIND LOAD ON LIVE LOAD:	WL	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	N	<-- N/A For Wall Design
BR, BREAKING LOAD :	BR	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	N	<-- N/A For Wall Design
TU, THERMAL FORCE:	TU	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	N	<-- N/A For Wall Design
EQ, SEISMIC LOAD ON SUPERSTRUCTURE:	EQ	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	N	<-- N/A For Wall Design
CT, VEHICLE COLLISION LOAD	CT	0.00	0.00	Distance above top pf wall equal to the height of rail	Include =	N	<-- N/A For Wall Design

Note: Per AASHTO 11.5.1, abutments and retaining walls should be designed for EH, WA, LS, DS, DC, TU, EQ. Therefore, including wind and breaking forces is conservative. Say OK

CANTILEVER WALL DESIGN -INPUT

General Information

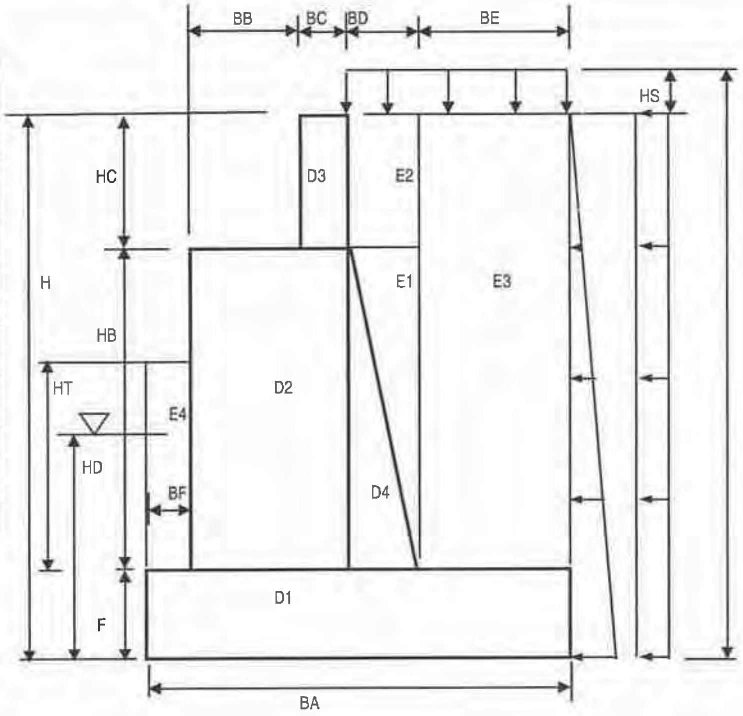
Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Abutment Geometry

Input Section : 4.0

CALCULATION OF WALL AND BACKFILL GEOMETRY:

		Prelim Size	User Adjust	Final Size (ft)	Approx Size (mm)
HEIGHT OF ABUTMENT / WALL, H:	H =	27.060	0.00	27.06	8300
HEIGHT OF FOOTING, F:	F =	3.280	0.00	3.28	1000
HEIGHT OF STEM, HB:	HB =	23.780	0.00	23.78	7300
HEIGHT OF BACKWALL, HC:	HC =	0.000	0.00	0.00	0
HEIGHT OF HIGH WATER, HD:	HD =	21.878	-1.01	20.87	6400
HEIGHT OF SURCHARGE, HS:	HS =	0.000	0.00	0.00	0
WIDTH OF FOOTING, BA:	BA =	15.080	0.00	15.08	4600
WIDTH OF BRIDGE SEAT, BB:	BB =	1.970	0.00	1.97	610
WIDTH OF BACKWALL, BC:	BC =	0.000	0.00	0.00	0
WIDTH OF BATTER OF STEM, BD:	BD =	1.970	1.62	3.59	1100
WIDTH OF FOOTING HEEL, BE:	BE =	7.525	0.00	7.53	2300
WIDTH OF FOOTING TOE, BF:	BF =	1.995	0.00	2.00	610
HEIGHT OF SOIL OVER TOE, HT:	HT =	9.020	0.00	9.02	2750
HEIGHT OF SOIL OVER HEEL, HH:	HH =	23.630	0.00	23.63	7300
HEIGHT OF SOIL AT BACKFACE FACE (HEEL), HS1	Hss1 =	27.06		26.91	8300
HEIGHT OF SOIL AT FRONT FACE FACE (TOE), HS2	Hss2 =	12.3	0.00	12.3	3750



OVERALL QUANTITIES:

WEIGHT OF CONCRETE WALL/L.F.:	20.849	Kips per l.f.
CONCRETE QUANTITY / L.F.:	5.148	C.Y. per l.f.

SUMMARY OF QUANTITIES:

STEEL / L.F. =	763.221	LBS/L.F.
CONC. / L.F =	5.148	C.Y./L.F.

Geometry Check: Check Width: **ok**
Check Height: **ok**

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

References:
- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:
- This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).
- Wall Design **Khost Bridge Notes**

Calculate Dead Loads

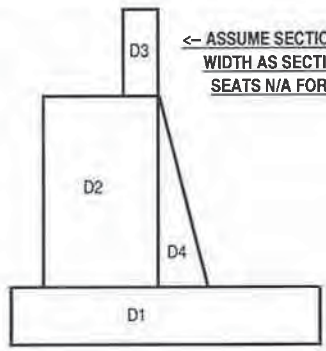
Primary Loads Section : 1.0

Superstructure Loads:

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	Superstructure	0.00	2.98	0.00			
DW	Superstructure	0.00	2.98	0.00			

<-- N/A FOR WALL DESIGN

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.



<-- ASSUME SECTION D3 IS THE SAME WIDTH AS SECTION D2 - BRIDGE SEATS N/A FOR WALL DESIGN

Substructure Loads:

AREA #		Vertical: Volume (CF)	γ_{conc} (pcf)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	D1	49.46	150.00	7.42	7.54	55.94			
	D2	46.85	150.00	7.03	2.98	20.94			
	D3	0.00	150.00	0.00	3.97	0.00			
	D4	42.69	150.00	6.40	5.16	33.05			
	Subtotal Concrete			**20.85**		**109.93**			

<-- N/A FOR WALL DESIGN

Total Dead Load:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	**20.85**		**109.93**			
TOTAL DW (Super)	**0.00**		**0.00**			
TOTAL DC (Substr. Only - Construction)	**20.85**		**109.93**			

<--SUPERSTRUCTURE LOADS N/A FOR WALL DESIGN

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Earth Loads

Primary Loads Section : 2.0

Compute Horizontal Earth Pressure, EH:

Coulomb's Active Earth Pressure: (AASHTO 3.11.5.3)

PHI, $\phi f' =$	33.00	Degrees, Rad =	0.58
DELTA, $\delta =$	22.00	Degrees, Rad =	0.38
BETA, $\beta =$	0.00	Degrees, Rad =	0.00
THETA, $\theta =$	81.04	Degrees, Rad =	1.41
Γ (per AASHTO Eq. 3.11.5.3-2)=	2.98		
Ka (per AASHTO Eq. 3.11.5.3-1)=	**0.335**		

At-Rest Earth Pressure Coeff:

Ko = **0.455**

Earth Pressure Coefficient to be Used for Design: 'Active pressure coefficients shall be estimated using Coulomb Theory.

WALL ON LEDGE:	N	(Y OR N)
WALL ON PILES:	N	(Y OR N)
Wall Height:	27.06	ft
Earth pressure Type:	Ka	
Ke =	**0.335**	<===== **Governs.**

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Ko =	0.46	
Ka =	0.29	
Ke (geotech) =	**0.290**	<=== **Does not govern.**

Compute Lateral Earth Pressure:

Application of lateral earth pressure shall be per AASHTO Figure C3.11.5.3-1. This shows a different application for Gravity and Cantilever (semi-gravity) walls.
Note that the reduction in lateral earth pressures due to the water table is not included in this section. It is included in the WA (Bouyancy) section of this design.

THIS SECTION IS FOR CANTILEVER OR SEMI-GRAVITY WALLS ONLY

Cantilever (semi-gravity) Walls:

Load inclination from horizontal, min = $\phi/3$ =	11.00	degrees
Load inclination from horizontal, max = $\phi*2/3$ =	22.00	degrees
GAMMA =	148.00	pcf
H = Soil Height at Back face, Hss1	**26.91**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	**17.93**	kips
Arm for Horiz Load above BOF = H/3 =	8.97	ft
Arm for Vert Load from Toe = F =	15.08	ft

Consider minimum inclination for Sliding, Overturning and Bearing Pressure:

Vertical Component, Pav = Pa*sin(ϕ/3) =	3.42	klf
Horizontal Component, Pah = Pa*cos(ϕ/3) =	17.60	klf

Consider maximum inclination for Footing Heel Reinforcement:

Vertical Component, Pav = Pa*sin(ϕ*2/3) =	6.72	klf
Horizontal Component, Pah = Pa*cos(ϕ*2/3) =	16.63	klf

N/A ---> THIS SECTION IS FOR GRAVITY WALLS ONLY

Gravity Walls:

Load inclination from horizontal = $\delta + (90-\theta)$ =	30.96	degrees
GAMMA =	148.00	pcf
H =	**26.91**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	**17.93**	kips
Arm for Horiz Load above BOF = H/3 =	**8.97**	ft
Arm for Vert Load from Toe=(BF+BB+BC+BD*2/3) =	6.36	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Pav = Pa*sin($\delta+(90-\theta)$) =	9.22	klf
Horizontal Component, Pah = Pa*cos($\delta+(90-\theta)$) =	15.38	klf
Is the wall a Gravity Wall?	N	

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CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.1

Include Passive Earth Pressure	Y
Pp Factor	1

ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kp = Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	148.00	pcf
H = Hss2= Height of Soil at Front Face - 1'	11.30	ft
Lateral EQ Load, Pp = 1/2*γ*Kp*H^2=	29.58	klf > Pah -------> Use Pp = Pah ---------> P_p = 17.60 klf
Arm for Horiz Load above BOF = H/3 =	3.77	ft (AASHTO pg 11-112)

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.2

Horizontal Earth Pressure, EH:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EH: Pa	3.42	15.08	51.60	17.60	8.97	157.91
EH: Pp			0.00	-17.60	3.77	-66.29
EH (For all cases except heel reinforcement):	**3.42**	**15.08**	**51.60**	**0.00**	**12.74**	**0.05**
EH: Pa	6.72	15.08	101.31	16.63	8.97	149.15
EH: Pp			0.00			0.00
EH (For Heel Reinforcement):	**6.72**	**15.08**	**101.31**	**16.63**	**8.97**	**149.15**

<=== Note, Based on AASHTO Figure C11.5.6-1, both the vertical and horizontal components of EH should be included here because they carry the same load factor.

Vertical Earth Pressure, EV:

AREA #		Volume (CF)	γ_{SOIL} (pcf)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EV	E1	42.69	148.00	6.32	6.36	40.17			
	E2	0.00	148.00	0.00	5.76	0.00			
	E3	178.94	148.00	26.48	11.32	299.73			
	E4	17.99	148.00	2.66	1.00	2.66			
TOTAL EV				**35.46**		**342.55**			

Note, **per AASHTO 11.6.1.2**, the weight of the soil over the battered portion of the stem or over the base of a footing may be considered as part of the effective weight of the abutment. This is consistant with design.

Earth Surcharge, ES: (This applies for construction case only)

q =	250.00	psf
Uniform Load on Wall, p=Ke*q =	0.084	ksf
Wall Height, H =	27.06	Feet
Heel Length, BE =	7.53	Feet
Footing Width, BA =	15.08	Feet
Wall Length Considered =	1.00	ft

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
ES	$P_{con}(h)$ = p*H*Length =				2.26	13.53	30.63
	$P_{con}(v)$ = q*BE*Length =	2.78	9.52	26.46			
TOTAL ES		**2.78**		**26.46**	**2.26**		**30.63**

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CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Live Loads

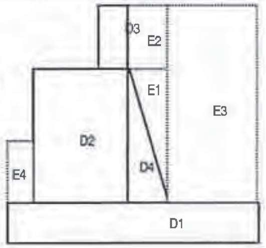
Primary Loads Section : 3.0

Superstructure Loads:

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LL+IM+PL	**Superstructure**	**0.00**	**2.98**	**0.00**			
BR	**Superstructure**				**0.000**	**27.06**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

<--SUPERSTRUCTURE LOADS N/A FOR WALL DESIGN



<-- ASSUME SECTION D3 IS THE SAME WIDTH AS SECTION D2 - BRIDGE SEATS N/A FOR WALL DESIGN

Live Load Surcharge Loads: LS

Per AASHTO 3.11.6.4, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. If the surcharge is for highway, the intensity of the load shall be consistent with provisions of Article 3.6.1.2. See Tables 3.11.6.4-1 and 3.11.6.4-2 for equivalent heights.

Compute Horizontal Live Load Surcharge: (To be used for bearing pressure and sliding load cases):

Ke =	0.335	
Unit Weight of Soil, γ =	148.000	pcf
Surcharge Height, heq =	0.00	Feet
LS(h) =(Ke)(γ)(heq)*H =	0.00	kips
Moment arm = H/2 =	13.53	kips

Compute Vertical Live Load Surcharge: (To be used for bearing pressure cases only):

LS(v) =(γ)(heq)(BD+BE) =	0.00	kips
Moment arm = Ba-(BD+BE)/2 =	9.52	kips

Compute Vertical Live Load Surcharge: (To be used for heel reinf cases only):

LS(v) =(γ)(heq)(BE) =	0.00	kips
Moment arm (to back of batter) = BE/2 =	3.76	kips

Live Load Surcharge, LS: Summary

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LS	**LS(v)**	**0.00**	**9.52**	**0.00**			
	LS(h)				**0.00**	**13.53**	**0.00**

Total Live Load Load:

AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL LL+IM+PED+BR+LS	**0.00**		**0.00**	**0.00**		**0.00**
TOTAL LL+IM+PED+BR+LS (Sliding Only)	**0.00**		**0.00**	**0.00**		**0.00**
TOTAL LS (Heel Reinf Only)	**0.00**	**3.76**	**0.00**			

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Water load (Buoyancy Forces)

Primary Loads Section : 4.0

HEIGHT OF STEM AT HIGH WATER:	17.59	
HEIGHT OF FOOTING AT HIGH WATER:	3.28	
WIDTH OF FOOTING, BA	15.08	
SOIL WEIGHT - WATER WEIGHT	85.60	pcf
UPWARD BOUYANT FORCE	-62.40	pcf

INCLUDE HORIZONTAL FORCE? Y

Horizontal Force = B(h) = (γ-(γ-62.4))*Ka)H^2/2, acts at HD/3:

Bouyant Load, WA:

AREA #		VOLUME (CF)	GAMMA (#/CF)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WA	B1 (Ftg)	49.46	-62.40	-3.09	7.54	-23.27			
	B2 (Stem)	97.79	-62.40	-6.10	2.98	-18.18			
	B3 (Soil over Ftg)	167.43	-62.40	-10.45	11.32	-118.24			
	STATIC						0.00	6.96	0.00
	SEISMIC						0.00	6.96	0.00
TOTAL WA (BL) (Static)				**-19.64**		**-159.70**	**0.00**		**0.00**
TOTAL WA (BL) (Seismic)				**-19.64**		**-159.70**	**0.00**		**0.00**

Calculate Stream Flow Pressure

Primary Loads Section : 4.1

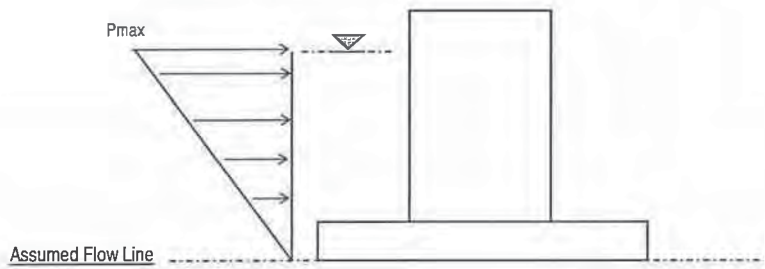
Note: The flow line is conservatively assumed to act at the bottom of the footing

Pmax:	0.0000	ksf
APPLIED:	N	

Force = 0.5 * Pmax * HD

Arm = HD * (2/3)

LOAD	HORIZONTAL: **FORCE (Kips)**	**ARM (Feet)**	**MOM (Ft x K)**
WA (SF)	0.00	13.91	0.00



Calculate Water Load & Stream Flow Load WA

Primary Loads Section : 4.2

Water Load (Bouyancy) & Stream Flow, WA:

AREA #	VOLUME (CF)	GAMMA (#/CF)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL WA (Static)			-19.64		-159.70	0.00		0.00
TOTAL WA (Seismic)			-19.64		-159.70	0.00		0.00

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Wind Loads

Primary Loads Section : 5.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WS	**Superstructure**				**0.00**	**27.06**	**0.00**
WL	**Superstructure**				**0.00**	**27.06**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Calculate Temperature Loads

Primary Loads Section : 6.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TU	**Superstructure**				**0.00**	**27.06**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Seismic Forces

Primary Loads Section : 7.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	Superstructure				0.000	27.06	0.00

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads:
(Ref: AASHTO Mononobe-Okabe Analysis.)

GAMMA = unit weight of soil =	148.00	Lbs/CF		
H = height of soil face =	27.06	Feet		
PHI = angle of internal friction of soil =	33.00	Degrees =	0.58	Radians
DELTA = angle of friction between soil & abut =	22.00	Degrees =	0.38	Radians
i = backfill slope angle =	0.00	Degrees =	0.00	Radians
BETA = slope of wall to the vertical	0.00	Degrees =	0.00	Radians

A =	0.29			
kh = horizontal acceleration coefficient	0.145	Consider Cohesion?	N	------> kh = a * 0.5, Wall is NOT Restrained from Horizontal Movement
kv = vertical acceleration coefficient	0.000			
THETA = arc tan (kh/(1-Kv) =	8.25	Degrees =	0.14	Radians
Kae (per AASHTO Eq. A11.1.1.1-2) =	**0.363**	**<===== Governs.**		

<u>Earth Pressure Coefficients to be Used for Design per Geotechnical Report:</u>
Kae (geotech) = **0.000** **<=== Does not govern.**

N/A
NOT GIVEN IN
GEOTECH REPORT

Load inclination from horizontal = δ =	22.00	degrees
Lateral EQ Load, Eae = 1/2*γ*Kae*H^2*(1-kv) =	19.67	klf
Arm for Horiz Load above BOF = H/3 =	9.02	ft (AASHTO pg 11-112)
Arm for Vert Load from Toe = BA =	15.08	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Eav = Eae*sin(δ) =	7.37	klf	Include EQ In Design =	Y
Horizontal Component, Eah = Eae*cos(δ) =	18.24	klf	EQ Factor =	1

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Seismic Forces

Primary Loads Section : 7.1

Include Seismic Passive Earth Pressure	Y	
Epe Factor	1	
kh = horizontal acceleration coefficient	0.145	
ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kpe = Seismic Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	148.00	pcf
Hff = Height of Soil at Front Face -1'	11.30	ft
Lateral EQ Load, Epe = 1/2*γ*Kpe*H^2=	29.58	klf ---> Equation A11.4-4
Arm for Horiz Load above BOF = Hff/3 =	3.77	ft (AASHTO pg 11-112)

SECTION 11: WALLS, ABUTMENTS, AND PIERS 11-117

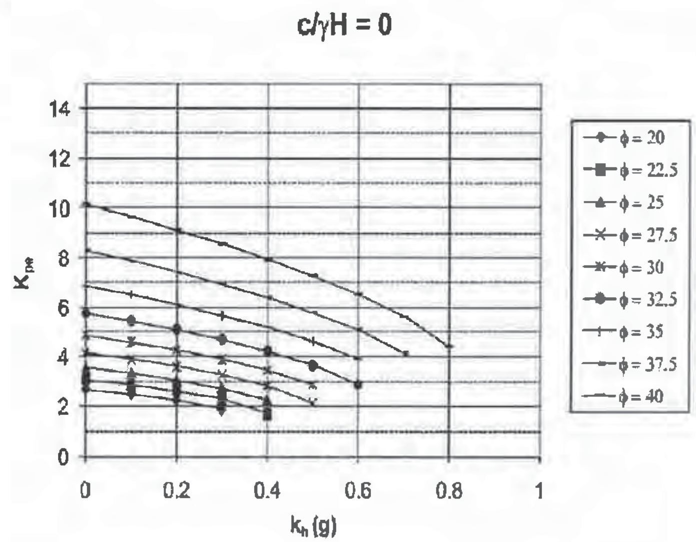


Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{h0}$ for wall heights greater than 20 ft.

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Seismic Forces Continued..

This section is not applicable for the wall design analysis.

Primary Loads Section : 7.2

WALL INERTIA EFFECTS

Per **AASHTO DIV 1A 6.4.3**, seismic design should take into account forces arising from seismically inducd lateral earth pressures (as computed above), additional forces arising from wall inertia and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely.

The following table computes the inertia forces due to the weight of the concrete and backfill.

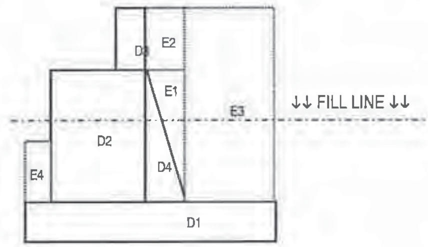
kh = 0.145

AREA #		DL (Kips)	DL*kh (Kips)	ARM (Feet)	MOM (Ft x K)
DL Wall	D1	7.42	1.08	1.64	1.76
	D2	7.03	1.02	15.17	15.46
	D3	0.00	0.00	27.06	0.00
	D4	6.40	0.93	11.21	10.40
	Subtotal	**20.85**	**3.02**	**9.14**	**27.63**
DL Backfill	E1	6.32	0.92	19.13	17.53
	E2	0.00	0.00	27.06	0.00
	E3	147.00	21.32	15.17	323.35
	E4	2.66	0.39	7.79	3.01
	Subtotal	**155.98**	**22.62**	**15.20**	**343.88**
TOTAL		**176.83**	**25.64**	**14.49**	**371.51**

FOR PIERS: Include DL above Fill Only

% of DL to be included

Area	%
D1	100%
D2	38%
D3	100%
D4	100% n/a
E1	100%
E2	100%
E3	100%
E4	100%



Total Seismic Loads, EQ:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	EQ Superstructure =				0.000	27.06	0.000
	Eae(v)	7.37	15.08	111.13			
	Eae(h)				18.24	9.02	164.53
	Epe(v)		15.08	0.00			
	Epe				-29.58	3.77	-111.40
	Fwi(h)				25.64	14.49	371.51
TOTAL EQ		**7.37**		**111.13**	**14.31**		**424.64**

% Eae(h) to be included:

100% **FOR PIERS: M-O ANALYSIS IS FOR RETAINED SOILS --> N/A FOR PIERS**

CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Calculate Vehicle Collision Loads

Primary Loads Section : 8.2

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
CT (Stem Design)	**Superstructure**				**0.00**	**0.00**	**0.00**
CT	**Superstructure**				**0.00**	**0.00**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

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CANTILEVER WALL DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Summary of Primary Loads

Primary Loads Section : 9.2

	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	20.85		109.93			
TOTAL DW (Super)	0.00		0.00			
TOTAL DC (Substr. Only - Construction)	20.85		109.93			
EH (For all cases except heel reinforcement):	3.42	15.08	51.60	0.00	12.74	0.05
EH (For Heel Reinforcement):	6.72	15.08	101.31	16.63	8.97	149.15
TOTAL EV	35.46		342.55			
TOTAL ES	2.78		26.46	2.26		30.63
TOTAL LL+IM+PED+BR+LS	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL LL+IM+PED+BR+LS (Sliding Only)	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL LS (Heel Reinf Only)	0.00	3.76	0.00	0.00	0.00	0.00
TOTAL WA (Static)	-19.64		-159.70	0.00		0.00
TOTAL WA (Seismic)	-19.64		-159.70	0.00		0.00
WS Superstructure				0.00	27.06	0.00
WL Superstructure				0.00	27.06	0.00
TU Superstructure				0.00	27.06	0.00
TOTAL EQ	7.37		111.13	14.31		424.64
CT (Stem Design)	0.00	0.00	0.00	0.00	0.00	0.00
CT	0.00	0.00	0.00	0.00	0.00	0.00

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes: This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Summary of Primary Loads

Load Combinations : 1.0

INCLUDE SEISMIC = Y

Load		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)	Notes	LRFD Load Combination Load Case
Dead Load	$DC_{SUB+SUPER}$	20.85	0.00	109.93	0.00	0.00	0.00	Super + Sub	
	DW	0.00	0.00	0.00	0.00	0.00	0.00	Super Only	
	DC_{SUB}	20.85	0.00	109.93	0.00	0.00	0.00	Sub Only - Construction	LC1 only
Earth Load	EH	3.42	15.08	51.60	0.00	12.74	0.05	All cases except Heel	Used in all load cases
	EH	6.72	15.08	101.31	16.63	8.97	149.15	For Heel Reinforcement	Not used in any load case
	EV	35.46	0.00	342.55	0.00	0.00	0.00		
Earth Load Surcharge	ES	2.78	0.00	26.46	2.26	0.00	30.63		
Live Load Surcharge	LS(v)	0.00	9.52	0.00	0.00	0.00	0.00		
	LS(h)	0.00	0.00	0.00	0.00	13.53	0.00		
Live Load	LL+IM+PED+BR+LS	0.00	0.00	0.00	0.00	0.00	0.00		
	LL+IM+PED+BR+LS	0.00	0.00	0.00	0.00	0.00	0.00	No LS for Sliding LC	LC4, LC8 & LC10
	LS	0.00	3.76	0.00	0.00	0.00	0.00		
Bouyant Load & Stream Force	WA	-19.64	0.00	-159.70	0.00	0.00	0.00	Static	
	WA	-19.64	0.00	-159.70	0.00	0.00	0.00	Seismic	LC9 &LC10
Wind Load	WS	0.00	0.00	0.00	0.00	27.06	0.00		
	WL	0.00	0.00	0.00	0.00	27.06	0.00		
Temperature Load	TU	0.00	0.00	0.00	0.00	27.06	0.00		
Seismic Load	EQ	7.37	0.00	111.13	14.31	0.00	424.64		
Vehicle Collision Load	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stem Wall	LC11 & LC12
	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stability	

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Limit States and Load Factors

Load Combinations : 2.0

Service Limit State

Per AASHTO 10.5.2, foundation design at the service limit state shall include settlements, horizontal movements, overall stability (of earth slopes) and scour at the design flood.
* These items are part of the geotechnical scope and are therefore NOT included in this design.

Strength Limit States

Per AASHTO 10.5.3, foundation design at the strength limit strength shall include structural resistance, scour, nominal bearing resistance, overturning or excessive loss of contact, sliding and constructability.
* These items, except scour, are addressed in this design.

Extreme Events Limit States

Per AASHTO 10.5.4, foundation shall be designed for extreme events such as a seismic event and vehicle collision.
* These items are addressed in this design.

Computation of the Load Modification Factor, h_i:

		Extreme	Strength
h_D Ductility Factor, (AASHTO 1.3.3):		1.00	1.00
h_R Redundancy Factor, (AASHTO 1.3.4):		1.00	1.00
h_I Operational Importance Factor, (AASHTO 1.3.5):		1.00	1.00
h_i (for loads for which y_i(max) is appropriate) (AASHTO Eq 1.3.2.1-2):	$h_i = h_D h_R h_I \geq 0.95$	1.00	1.00
h_i (for loads for which y_i(min) is appropriate) (AASHTO Eq 1.3.2.1-3):	$h_i = 1 / h_D h_R h_I \leq 1.00$	1.00	1.00

Since these factors are 1.0, they have not yet been incorporated into the design template.

h_D Ductility Factor (for all other limit states $h_D = 1.00$)

- $h_D \geq$ 1.05 for nonductile components and connections.
- $h_D =$ 1.00 for conventional designs and details complying with the specifications.
- $h_D \geq$ 0.95 for components and connections for which additional ductility-enhancing

h_R Redundancy Factor (for all other limit states $h_R = 1.00$)

- $h_R \geq$ 1.05 for nonredundant members
- $h_R =$ 1.00 for conventional levels of redundancy
- $h_R \geq$ 0.95 for exceptinal levels of redundancy

h_I Operational Importance Factor

- $h_I \geq$ 1.05 for a bridge of operational importance
- $h_I =$ 1.00 for typical bridges
- $h_I \geq$ 0.95 for relatively less important bridges

Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2), g_p:

	Maximum	Minimum	
DC (Dead Load, General):	1.25	0.90	
DW (Wearing Surface & Utilities):	1.50	0.65	
EH (Horiz Earth):	1.43	0.90	<-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.
ES (Horiz Earth):	1.50	0.75	
EV (Vertical Earth, Retaining Structure):	1.35	1.00	

Live Load Factor During a Seismic Event, g_{EQ}:

	Maximum	Minimum	
g_{EQ} (AASHTO C3.4.1):	0.50	0.00	<--- **Seismic Included**

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Load Combinations : 3.0

NOTES:

1. Load Combination Strength II does not need to be checked since it applies to special design vehicles.
2. Load Combination Strength III does not need to be checked during construction since WS is not a significant load.
3. Load Combination Strength IV does not need to be checked since it applies to bridges with very high dead load to live load ratios.
4. Load Combination Strength V does not need to be checked during construction since WS and WL are not significant loads.
5. Extreme Event load combinations do not need to be checked during construction.
6. Extreme Event II load combinations does not need to be checked for abutments.
7. Service limit state load combinations do not need to be checked for abutment stability / reinforcement.
8. Fatigue limit state load combinations do not need to be checked for abutment stability / reinforcement.
9. All remaining load cases shall be checked using load factors which would provide max effect for either bearing or sliding / eccentricity similar to AASHTO Figures C11.5.5-1 and C11.5.5.2.
10. Bouyancy has been included in sliding load combinations. A load factor of 0.0 has been used for bearing pressure load combinations since it is conservative to ignore sliding for these computations.

Strength	LC1	LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): gp max*(DCsub)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Strength	LC2	LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Bearing	LC3	LC3 - STRENGTH I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Sliding	LC4	LC4 - STRENGTH I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Bearing	LC5	LC5 - STRENGTH III BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Sliding	LC6	LC6 - STRENGTH III SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Bearing	LC7	LC7 - STRENGTH V BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Sliding	LC8	LC8 - STRENGTH V SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Extreme Bearing	LC9	LC9 - EXTREME EVENT I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+gEQ MAX*(LL+IM+PL+BR+LS)+1.0*(EQ)
Extreme Sliding	LC10	LC10 - EXTREME EVENT I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+gEQ MIN*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)
Extreme Bearing	LC11	LC11 - EXTREME EVENT II BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(CT)
Extreme Sliding	LC12	LC12 - EXTREME EVENT II SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(CT)

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

LRFD Load Combinations

Load Combinations : 3.1

LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): $q_{p\,max}$*(DCsub)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC_{SUB}	1.25	26.06		137.41	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
ES	1.50	4.17		39.69	3.40		45.95
SUM		**82.98**		**713.09**	**3.40**		**46.02**

LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	26.06		137.41	0.00		0.00
DW	1.5	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
ES	1.50	4.17		39.69	3.40		45.95
SUM		**82.98**		**713.09**	**3.40**		**46.02**

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.2

LC3 - STRENGTH I BEARING: $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	26.06		137.41	0.00		0.00
DW	1.5	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
LL+IM+PL+BR+LS	1.75	0.00		0.00	0.00		0.00
WA	1.00	-19.64		-159.70	0.00		0.00
TU	0.50	0.00		0.000	0.0000		0.000
SUM		**59.18**		**513.70**	**0.01**		**0.08**

LC4 - STRENGTH I SLIDING: $q_{p\,min}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,min}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	18.76		98.94	0.00		0.00
DW	0.65	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.00	35.46		342.55	0.00		0.00
LL+IM+PL+BR+LS	1.75	0.00		0.00	0.00		0.00
WA (static)	1.00	-19.64		-159.70	0.00		0.00
TU	0.50	0.00		0.00	0.000		0.000
SUM		**39.47**		**355.33**	**0.01**		**0.08**

LC5 - STRENGTH III BEARING: $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	26.06		137.41	0.00		0.00
DW	1.5	0.00		0.00	0.00		0.00
EH	1.425	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
WA (static)	1.00	-19.64		-159.70	0.00		0.00
WS	1.40	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**59.18**		**513.70**	**0.01**		**0.08**

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.3

LC6 - STRENGTH III SLIDING: $g_{p\ min}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ min}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.90	18.76		98.94	0.00		0.00
DW	0.65	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.00	35.46		342.55	0.00		0.00
WA	1.00	-19.64		-159.70	0.00		0.00
WS	1.40	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**39.47**		**355.33**	**0.01**		**0.08**

LC7 - STRENGTH V BEARING: $g_{p\ max}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ max}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	26.06		137.41	0.00		0.00
DW	1.5	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
LL+IM+PL+BR+LS	1.35	0.00		0.00	0.00		0.00
WA	1.00	-19.64		-159.70	0.00		0.00
WS	0.40	0.00		0.00	0.00		0.00
WL	1.00	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**59.18**		**513.70**	**0.01**		**0.08**

LC8 - STRENGTH V SLIDING: $g_{p\ min}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ min}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	18.76		98.94	0.00		0.00
DW	0.65	0.00		0.00	0.00		0.00
EH	1.425	4.88		73.53	0.01		0.08
EV	1	35.46		342.55	0.00		0.00
LL+IM+PL+BR+LS	1.35	0.00		0.00	0.00		0.00
WA	1.00	-19.64		-159.70	0.00		0.00
WS	0.40	0.00		0.00	0.00		0.00
WL	1.00	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**39.47**		**355.33**	**0.01**		**0.08**

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC9 - EXTREME EVENT I BEARING: $q_{p\,max}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,max}$*(EV)+$q_{EQ\,MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	26.06		137.41	0.00		0.00
DW	1.5	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
LL+IM+PL+BR+LS	0.50	0.00		0.00	0.00		0.00
WA	0.00	0.00		0.00	0.00		0.00
EQ	1.00	7.37		111.13	14.31		424.64
SUM		**86.18**		**784.53**	**14.31**		**424.71**

LC10 - EXTREME EVENT I SLIDING: $q_{p\,min}$*(DC+DW)+$q_{p\,max}$*(EH)+$q_{p\,min}$*(EV)+$q_{EQ\,MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	18.76		98.94	0.00		0.00
DW	0.65	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.00	35.46		342.55	0.00		0.00
LL+IM+PL+BR+LS	0.00	0.00		0.00	0.00		0.00
WA (seismic)	1.00	-19.64		-159.70	0.00		0.00
EQ	1.00	7.37		111.13	14.31		424.64
SUM		**46.84**		**466.46**	**14.31**		**424.71**

CANTILEVER WALL DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

LRFD Load Combinations Cont.

Load Combinations : 3,4

LC11 - EXTREME EVENT II BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$g_{EQ\,MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	26.06		137.41	0.00		0.00
DW	1.5	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.35	47.88		462.45	0.00		0.00
LL+IM+PL+BR+LS	0.50	0.00		0.00	0.00		0.00
WA	0.00	0.00		0.00	0.00		0.00
CT	1.00	0.00		0.00	0.00		0.00
SUM		**78.81**		**673.40**	**0.01**		**0.08**

LC12 - EXTREME EVENT II SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+$g_{EQ\,MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	18.76		98.94	0.00		0.00
DW	0.65	0.00		0.00	0.00		0.00
EH	1.43	4.88		73.53	0.01		0.08
EV	1.00	35.46		342.55	0.00		0.00
LL+IM+PL+BR+LS	0.50	0.00		0.00	0.00		0.00
WA (seismic)	1.00	-19.64		-159.70	0.00		0.00
CT	1.00	0.00		0.00	0.00		0.00
SUM		**39.47**		**355.33**	**0.01**		**0.08**

CANTILEVER WALL DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes: **This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).**

Check Bearing Resistance (per AASHTO 11.6.3.2) -- ON SOIL

Stability : 1.0

If supported on soil, the vertical stress (σ_v) shall be calculated assuming a uniformly distributed pressure (V) over an effective base area (B-2e). AASHTO Fig 11.6.3.2-1 —> qr / Φβ = q_n =

If supported on rock, the vertical stress (σ_v) shall be calculated assuming a linearly distributed pressure over an effective base area. AASHTO Fig 11.6.3.2-2 —> qr / Φβ = q_n =

Factored Bearing Resistance, qr: qr = Φβ * q_n = 17.78 ksf <— **Note per Geotech, this is factored net bearing resistance**

Strength Bearing Resistance Factor, Φβ (AASHTO Table 11.5.7-1): 0.45 qr = Φβ * q_n = 17.78 ksf —> qr / Φβ = qn = 8.00 ksf

Extreme Event Bearing Resistance Factor, Φβ (AASHTO 10.5.5.3.3): 1.00 qr = Φβ * q_n = 17.78 ksf —> qr / Φβ = qn = 17.78 ksf

Note --> See AASHTO Table 11.5.7-1 to determine Φβ Factor

	LOAD COMBINATION	Vertical Force (Kips)	Resisting Moment (Ft x K)	Overturn Moment (Ft x K)	Mnet (Ft x K)	Eccentricty from Toe, et=Mnet/V (Ft)	Eccentricty from CL, e=B/2-et (Ft)	σ_v on soil (ksf)	$\sigma_{v\,max}$ on rock (ksf)	$\sigma_{v\,min}$ on rock (ksf)	$\sigma_v < \Phi\beta^* q_n$	
Strength	LC1	82.98	713.09	46.02	667.06	8.04	-0.50	5.16	4.41	6.59	OK	
Strength	LC2	82.98	713.09	46.02	667.06	8.04	-0.50	5.16	4.41	6.59	OK	
Bearing	LC3	59.18	513.70	0.08	513.62	8.68	-1.14	3.41	2.15	5.70	OK	
Sliding	LC4	39.47	355.33	0.08	355.25	9.00	-1.46	2.19	1.10	4.14	OK	<-*N/A Sliding Combination
Bearing	LC5	59.18	513.70	0.08	513.62	8.68	-1.14	3.41	2.15	5.70	OK	
Sliding	LC6	39.47	355.33	0.08	355.25	9.00	-1.46	2.19	1.10	4.14	OK	<-*N/A Sliding Combination
Bearing	LC7	59.18	513.70	0.08	513.62	8.68	-1.14	3.41	2.15	5.70	OK	
Sliding	LC8	39.47	355.33	0.08	355.25	9.00	-1.46	2.19	1.10	4.14	OK	<-*N/A Sliding Combination
Ex. Bearing	LC9	86.18	784.53	424.71	359.82	4.17	3.37	10.32	13.76	0.00	OK	
Ex. Sliding	LC10	46.84	466.46	424.71	41.75	0.89	6.65	26.27	35.03	0.00	N/A	<-*N/A Ex. Sliding Combination
Ex. Bearing	LC11	78.81	673.40	0.08	673.32	8.54	-1.00	4.61	3.14	7.31	OK	
Ex. Sliding	LC12	39.47	355.33	0.08	355.25	9.00	-1.46	2.19	1.10	4.14	OK	<-*N/A Ex. Sliding Combination

*** Sliding Load Combinations are Not Applicable for checking the Bearing**

CANTILEVER WALL DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Check Overturning (per AASHTO 11.6.3.3) -- ON SOIL

Stability : 2.0

e allowable (ftgs on soil): 3.77 ft
e allowable (ftgs on rock): 5.66 ft
If e < e allowable, Overturning is OK:

	LOAD COMBINATION	Eccentricty from CL, e=B/2-et (Ft)	Check Overturning	
Strength	LC1	-0.50	OK	
Strength	LC2	-0.50	OK	
Bearing	LC3	-1.14	OK	
Sliding	LC4	-1.46	OK	<--*N/A Sliding Combination
Bearing	LC5	-1.14	OK	
Sliding	LC6	-1.46	OK	<--*N/A Sliding Combination
Bearing	LC7	-1.14	OK	
Sliding	LC8	-1.46	OK	<--*N/A Sliding Combination
Ex. Bearing	LC9	3.37	OK	
Ex. Sliding	LC10	6.65	N/A	<--*N/A Ex. Sliding Combination
Ex. Bearing	LC11	-1.00	OK	
Ex. Sliding	LC12	-1.46	OK	<--*N/A Ex. Sliding Combination

*** Sliding Load Combinations are Not Applicable for checking Overturning**

CANTILEVER WALL DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Check Sliding (per AASHTO 10.6.3.4)

Stability : 3.0

Ignore Passive Resistance of Soil per MassHighway

Strength Sliding Resistance Factor, $\Phi\tau$ (AASHTO Table 11.5.7-1):	1.00	
Extreme Event Sliding Resistance Factor, $\Phi\varpi$ (AASHTO 10.5.5.3.3):	1.00	
Internal Friction Angle of Drained Soil, Φ_f:	33.00	degrees
tan δ: = tan Φ_f (per AASHTO 10.6.3.4-2):	0.65	for concrete against soil. Multiply by 0.8 for precast concrete footing

	LOAD COMBINATION	Vertical Force (Kips)	Rt = V * tan δ: (Kips)	$\Phi\tau$ (Strength) $\Phi\omega$ (Extreme) (Kips)	Nom. Sliding Resistance $\Phi\tau$*Rt (Kips)	Horiz Force (Kips)	Check Sliding	
Strength	LC1	82.98	53.89	1.00	53.89	3.40	OK	<--*N/A Strength Combination
Strength	LC2	82.98	53.89	1.00	53.89	3.40	OK	<--*N/A Strength Combination
Bearing	LC3	59.18	38.43	1.00	38.43	0.01	OK	<--*N/A Bearing Combination
Sliding	LC4	39.47	25.63	1.00	25.63	0.01	OK	
Bearing	LC5	59.18	38.43	1.00	38.43	0.01	OK	<--*N/A Bearing Combination
Sliding	LC6	39.47	25.63	1.00	25.63	0.01	OK	
Bearing	LC7	59.18	38.43	1.00	38.43	0.01	OK	<--*N/A Bearing Combination
Sliding	LC8	39.47	25.63	1.00	25.63	0.01	OK	
Ex. Bearing	LC9	86.18	55.97	1.00	55.97	14.31	OK	<--*N/A Ex. Bearing Combination
Ex. Sliding	LC10	46.84	30.42	1.00	30.42	14.31	OK	
Ex. Bearing	LC11	78.81	51.18	0.65	33.24	0.00	OK	<--*N/A Ex. Bearing Combination
Ex. Sliding	LC12	39.47	25.63	0.65	16.65	0.00	OK	

Results Summary:

Stability : 4.0

STABILITY RESULTS:

LOAD COMBINATION:	BEARING RESISTANCE	OVERTURNING	SLIDING	
LC1	OK	OK	OK	<== Construction
LC2	OK	OK	OK	<== Construction
LC3	OK	OK	OK	
LC4	OK	OK	OK	
LC5	OK	OK	OK	
LC6	OK	OK	OK	
LC7	OK	OK	OK	
LC8	OK	OK	OK	
LC9	OK	OK	OK	
LC10	N/A	N/A	OK	
LC11	OK	OK	OK	
LC12	OK	OK	OK	

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).
Wall Design

Design Parameters

Reinforcement : 1.0

GEOMETRY

H of Footing, h :	3.28	ft
bw (per linear ft of wall) :	12.00	in

MATERIAL PROPERTIES

Compressive Strength: f'c :	4.00	ksi	
Min Yield Strength: fy :	60.00	ksi	
Max. Agg. Size :	1.50	in	
Es :	29000	ksi	AASHTO 5.4.3.2
Tension Reinforcement Strain: ε_s :	0.002	ε_s = fy / Es	
β :	1.881		AASHTO EQ 5.8.3.4.2-1

Design Heel and Toe Reinforcement

Reinforcement : 2.1

FACTORED HEEL DESIGN LOADS	Load Factor, γ_p AASHTO Table 3.4.1-2	Vertical Force & Design Shear (Kips)	Arm (Feet)	Design Moment (Ft x K)
DC (Heel Concrete)	1.25	4.63	3.76	17.41
EV (Heel Soil)	1.35	35.75	3.76	134.52
EH (Vertical Component)	1.43	9.57	7.53	72.04
LS	1.75	0.00	3.76	0.00
SUM		**49.95**		**223.97**

* See load combs, Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2) for the above Load Factors

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.2

Footing Toe Width, BF: 2.00 ft

FACTORED TOE DESIGN LOADS LOAD COMBINATION	σ_v Factored Toe Pressure (ksf)	Factored Toe Shear (Kips)	Factored Toe Moment (Ft x K)
LC1	5.16	10.30	10.27
LC2	5.16	10.30	10.27
LC3	3.41	6.80	6.78
LC4	2.19	4.37	4.36
LC5	3.41	6.80	6.78
LC6	2.19	4.37	4.36
LC7	3.41	6.80	6.78
LC8	2.19	4.37	4.36
LC9	10.32	20.59	20.54
LC10	n/a	0.00	0.00
MAX		**20.59**	**20.54**

Note: Based on AASHTO 10.6.5, the structural design of an eccentrically loaded foundation can assume a triangular or eccentrically loaded area. This spreadsheet conservatively assumes a uniform pressure of sv max over the toe of the footing. Based on AASHTO Figure C5.13.3.6.1-1, The toe and heel shear can be computed at a distance dv from the face of support. This spreadsheet computes it at the support, which is conservative.

10.6.5—Structural Design

The structural design of footings shall comply with the requirements given in Section 5.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions.

FOOTING HEEL REINF (TOP BARS):

USE #	**8.00**	@	**4.00 IN**
Abar =	0.79	in^2	
dbar =	1.00	in	
Asprov =	2.37	in^2	

FOOTING TOE REINF (BOTTOM BARS):

USE #	**6.00**	@	**6.00 IN**
Abar =	0.44	in^2	
dbar =	0.75	in	
Asprov =	0.88	in^2	

CRITICAL SECTION FOR WALLS

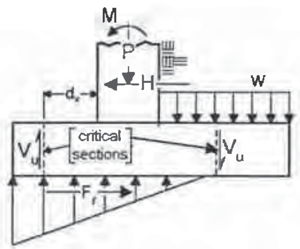

Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

CRITICAL SECTION FOR ABUTMENTS

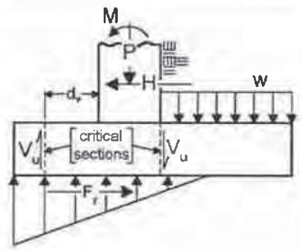

Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

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CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.3

CHECK FLEXURAL RESISTANCE	HEEL	TOE	AASHTO 5.7, 5.7.2.2, 5.7.3.2, 5.7.3.2.2	
Factored Moment, Mu =	223.97	20.54	k*ft	
Resistance Factor, phi: Φ =	0.90	0.90		AASHTO 5.5.4.2
Assume Cover, dc =	2.00	3.00	in	ACI 318-08 - 7.7
Shear Depth: ds =	36.86	35.99	in = h - cover - 1/2db(main)	
Depth of Equivalent Stress Block: a =	3.49	1.29	in = c*β1 = Asfy/0.85f'cb	AASHTO Eq. 5.7.3.1.1-4
Nominal Flexural resistance, Mn =	416.14	155.49	kip ft = [Asfy(ds-a/2)]/12	AASHTO Eq. 5.7.3.2.2-1
Factored Resistance, ΦMn =	374.53	139.94		AASHTO Eq. 5.7.3.2.1-1
As required for Mu:	0.79	0.09	in^2	
Flexure OK?	**OK**	**OK**		

CHECK MINIMUM REINFORCEMENT	HEEL	TOE	AASHTO 5.7.3.3.2	
Section Modulud: Sc =	3098.42	3098.42	in^3	
Compressive Strength: f'c =	4.00	4.00	ksi	
Modulus of Rupture: fr =	0.74	0.74	ksi = $0.37*(f'c)^{1/2}$	AASHTO 5.4.2.6
Cracking Moment: Mcr = Sc*fr =	191.07	191.07	kip ft	AASHTO Eq. 5.7.3.2.2-1
Factored Flexural Resistance: Mr1 = 1.2*Mcr =	229.28	229.28	kip ft	
Factored Moment, Mu =	223.97	20.54	k*ft	
Factored Flexural Resistance: Mr2 = 1.33*Mu =	297.88	27.32	kip ft	
Controlling Mr = min(Mr1, Mr2)	229.28	27.32	kip ft	
Factored Resistance, phi*Mn =	374.53	139.94		AASHTO Eq. 5.7.3.2.1-1
As required for Mr:	1.4227	0.1693	in^2	
As required for Temp Steel (#4@18"):	0.1333	0.1333	in^2	
As provided =	2.37	0.88	in^2	
Min Reinforcement OK?	**OK**	**OK**		

CHECK CRACK CONTROL BY DIST REINF.	HEEL	TOE	AASHTO 5.7.3.4, 5.10.3.1	
Exposure Factor: γ_e =	0.75	0.75	Class 2 Exposure	ASSHTO 5.7.3.4
β_s factor=	1.08	1.12	β_s factor = 1 + (dc / 0.7* (h-dc))	ASSHTO 5.7.3.4-1
f_{ss} =	36	36	ksi f_{ss} = .6*fy	
s_{max} =	8.55	8.05	in s_{max} < = 700 ge / βs fss	ASSHTO 5.7.3.4-1
s_{min} =	3.25	3.00	in s_{min} = max(1.5*db,1.5*agg,1.5")+db	AASHTO 5.10.3.1.1
SPACING OK?	**OK**	**OK**		

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CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Design Heel and Toe Reinforcement Cont.

Reinforcement : 2.4

CHECK SHEAR RESISTANCE	**HEEL**	**TOE**	**AASHTO 5.13.3.6, 5.8.3**		
Factored Shear Force, Vu =	49.95	20.59	kips		
Factored Moment, Mu =	223.97	20.54	k*in		
Es =	29000	29000			AASHTO 5.4.3.2
Resistance Factor, phi: Φ =	0.90	0.90			AASHTO 5.5.4.2
bw (per linear ft of wall) =	12.00	12.00	in		
Effective Depth: dv =	35.12	35.34	in	dv = max((ds-a/2),max(0.9ds, 0.72h))	AASHTO 5.8.2.9
H of Ftg, h:	39.36	39.36	in		
bw (per linear ft of ftg) =	12.00	12.00	in		
Area of Conc on Tension Side, Ac =	236.16	236.16	in	Ac = h*bw/2 =	
As (flexural) provd =	2.37	0.88	in^2		
Max. Size of Coarse Aggregate, ag =	1.50	1.50	in		
Mu min =	1754.26	727.66	k*in	Mu min = Vu*dv =	
Mu (controlling) =	1754.26	727.66	k*in		
Spg between top and bottom reinf, sx =	34.36	34.36	in		
Crack spg parameter, sxe =	22.26	22.26		sxe = 1.38*sx/(ag+0.63)	
Strain = ε_s=	0.0015	0.0016		ε_s=(Mu/dv+Vu)/(Es*As)	AASHTO EQ 5.8.3.4.2-4
Θ =	147.54	163.80		Θ = 29+3500*ε_s	AASHTO EQ 5.8.3.4.2-3
β =	1.91	1.81		β = 4.8/(1+750ε_s)*(51/(39+s_{xe}))	
Nom Shear Resistance, Vn1 =	421.41	424.06	kips	Vn1 = 0.25*f'c*bv*dv	AASHTO 5.8.3.3-2
Nominal Shear Resistance: Vn2 = Vc =	50.92	48.45	kips	Vn2 = Vc = 0.0316*β*f'c.5*bv*dv	AASHTO 5.8.3.3-3
Nom Shear Resistance, Vn =	50.92	48.45	kips	Vn = min (Vn1, Vn2)	
phi*Vn =	45.82	43.61			
Shear OK?	**OK**	**OK**			
Opposite Face Reinf As provd. =	0.88	2.37	in^2		
As min crack =	1.24	1.24	in^2	As min crack = 0.003*b*sx	
min (As front, back) > As min ?	**N/A**	**N/A**			

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CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Stem Reinforcement

Reinforcement : 3.0

1. Reinforcement does not need to be checked for construction loading since that is a temporary load case.
Check the stem reinforcement at various locations along the stem and at the base of the backwall.

Height of Stem plus Backwall, h = H - F =	23.78	ft
Height of Backwall =	0.00	ft
Ftg Dowel Lap Length:	7.00	ft
Width of Stem at the Base:	5.56	ft
Width of Backwall:	0.00	ft
Width of Batter:	3.59	ft

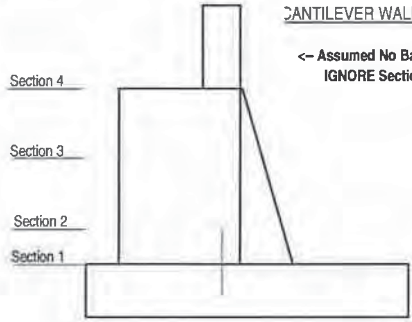
Section	Height of h	Height from top	Width Batter	Width conc	
1	1.00	23.78	3.59	5.56	<=== This section is at the bottom of the stem.
2	0.71	16.78	2.53	4.50	<=== This section is at the top of the footing dowel.
3	0.35	8.39	1.27	3.24	<=== This section is halfway in between top of footing dowel and top of batter.
4	0.00	0.00		0.00	<=== This section is at the base of the backwall

Horizontal Earth Pressure, EH at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	27.06				0.00	12.74	0.05
Top of Ftg	23.78				0.00	7.93	0.03
Top of Dowel	16.78				0.00	5.59	0.01
Mid-Height	8.39				0.00	2.80	0.00
Bot of Backwall	0				0.00	0.00	0.00

CANTILEVER WALL DESIGN

<-- Assumed No Backwall - IGNORE Section 4



Live Load Surcharge, LS at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	27.06				0.00	13.53	0.00
Top of Ftg	23.78				0.00	11.89	0.00
Top of Dowel	16.78				0.00	8.39	0.00
Mid-Height	8.39				0.00	4.20	0.00
Bot of Backwall	0				0.00	0.00	0.00

Seismic Load, EQ at Various Heights along Stem:

	Height from Top of Wall (Feet)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Original Calcs	27.06				14.31		424.64
Top of Ftg	23.78				12.57	11.89	149.47
Top of Dowel	16.78				8.87	8.39	74.43
Mid-Height	8.39				4.44	4.20	18.61
Bot of Backwall	0				0.00	0.00	0.00

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

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Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.1

Load Combination - STRENGTH I		At Top of Ftg		Top of Dowel		Mid-Height Abut		Bot of Backwall	
LOAD	Load Factor	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)
EH	1.43	0.00	0.04	0.00	0.01	0.00	0.00	0.00	0.00
LS	1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SUM		**0.00**	**0.04**	**0.00**	**0.01**	**0.00**	**0.00**	**0.00**	**0.00**

Load Combination - EXTREME EVENT I		At Top of Ftg		Top of Dowel		Mid-Height Abut		Bot of Backwall	
LOAD	Load Factor	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)	Horiz Force (Kips)	Overturn Moment (Ft x K)
EH	1.43	0.00	0.04	0.00	0.01	0.00	0.00	0.00	0.00
LS	0.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EQ	1.00	12.57	149.47	8.87	74.43	4.44	18.61	0.00	0.00
SUM		**12.58**	**149.51**	**8.87**	**74.44**	**4.44**	**18.61**	**0.00**	**0.00**

CHECK FLEXURAL RESISTANCE	**SECT 1**	**SECT 2**	**SECT 3**	**SECT 4**	**AASHTO 5.7**	
Section Height / Location =	23.78	16.78	8.39	0.00	ft	
Factored Moment, Mu =	149.51	74.44	18.61	0.00	k*ft	
Resistance Factor, phi: Φ =	0.90	0.90	0.90	0.90		AASHTO 5.5.4.2
H of Stem, h:	5.56	4.50	3.24	0.00	ft	
Cover, dc =	2.00	2.00	2.00	2.00	in	ACI 318-08: Sec 7.7.1
BAR # =	6.00	6.00	6.00	6.00		
SPACING =	6.00	8.00	8.00	8.00	in	
Main Abar =	0.44	0.44	0.44	0.44	in^2	
Main db =	0.750	0.750	0.750	0.750	in	
As provd. =	0.88	0.66	0.66	0.66	in^2	
Shear Depth: ds =	64.35	51.66	36.46	-2.38	in. = h - cover - 1/2db(main)	
Depth of Equivalent Stress Block: a =	1.29	0.97	0.97	0.97	in = c*β1 = Asfy/0.85f'cb	AASHTO 5.7.2.2, 5.7.3.2
Nominal Flexural resistance, Mn =	280.27	168.89	118.73	-9.44	kip ft = [Asfy(ds-a/2)]/12	AASHTO 5.7.3.2.2
Factored Resistance, phi*Mn =	252.24	152.00	106.86	-8.50		AASHTO Eq. 5.7.3.2.1-1
As required for Mu:	0.5248	0.3247	0.1152	0.0000	in^2	
Flexure OK?	**OK**	**OK**	**OK**	**N/A**		

<-- Assumed No Backwall - IGNORE Section 4

<-- 2" for Concrete exposed to earth or weather: No. 6 thru No 18 bars

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Wall Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.2

CHECK MINIMUM REINFORCEMENT	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.7.3.3.2	
Section Modulud: Sc =	8903.12	5840.37	3016.99	0.00	in^3	
Modulus of Rupture: fr =	0.74	0.74	0.74	0.74	ksi = $0.37*(f'c)^{1/2}$	AASHTO 5.4.2.6
Cracking Moment: Mcr = Sc*fr =	549.03	360.16	186.05	0.00	kip ft	
Factored Flexural Resistance: Mr1 = 1.2*Mcr =	658.83	432.19	223.26	0.00	kip ft	
Factored Moment, Mu =	149.51	74.44	18.61	0.00	k*ft	
Factored Flexural Resistance: Mr2 = 1.33*Mu =	198.85	99.00	24.75	0.00	kip ft	
Controlling Mr = min(Mr1, Mr2)	198.85	99.00	24.75	0.00	kip ft	
Factored Resistance, phi*Mn =	252.24	152.00	106.86	-8.50		AASHTO Eq. 5.7.3.2.1-1
As required for Mr:	0.6922	0.4285	0.1513	-3.2300	in^2	
As provided =	0.88	0.66	0.66	0.66	in^2	
Min Reinforcement OK?	**OK**	**OK**	**OK**	**N/A**		

<-- Assumed No Backwall - IGNORE Section 4

CHECK CRACK CONTROL BY DIST REINF.	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.7.3.4, 5.10.3.1	
Exposure Factor: γ_e =	0.75	0.75	0.75	0.75	Class 2 Exposure	ASSHTO 5.7.3.4
H of Stem, h:	66.72	54.04	38.84	0.00	in	
β_s factor=	1.04	1.05	1.08	-0.43	1 + (dc / 0.7* (h-dc))	ASSHTO 5.7.3.4-1
f_{ss} =	36	36.00	36.00	36.00	ksi = .6*fy	
s_{max} =	9.97	9.82	9.53	-38.03	in < = 700 γ_e / β_s f_{ss}	ASSHTO 5.7.3.4-1
Main db =	0.750	0.750	0.750	0.750	in	
s_{min} = max(1.5*db,1.5*agg,1.5")+db =	3.00	3.00	3.00	3.00	in	AASHTO 5.10.3.1.1
SPACING =	6.00	8.00	8.00	8.00	in	
SPACING OK?	**OK**	**OK**	**OK**	**N/A**		

<-- Assumed No Backwall - IGNORE Section 4

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

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Structure:	Wall Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.3

CHECK SHEAR TRANSFER	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.8.4.1, 5.8.4.3, 5.8.4.4
Cohesion Factor, c =	0.075	0.075	0.075	0.075	ksi, assumes CJ not intentionally roughened
Friction Factor, μ =	0.6	0.60	0.60	0.60	
Fraction of strength for interface shear, K1 =	0.2	0.20	0.20	0.20	
Limiting Interface Shear Resistance, K2 =	0.8	0.80	0.80	0.80	ksi
Lvi = H of Stem, h:	66.72	54.04	38.84	0.00	ft
bvi = bw (per linear ft of wall) =	12.00	12.00	12.00	12.00	in
Interface Area, Acv = Lvi*bvi =	800.64	648.47	466.07	0.00	in^2
Back Face (Flexural) As provd. =	0.88	0.66	0.66	0.66	in^2
Front Face (Dowels) As provd. =	0.44	0.44	0.44	0.44	in^2
Interface Reinf Provided, Avf = As back+front =	1.32	1.10	1.10	1.10	in^2
Vni = c*Acv+μ*Avf*Fy =	107.57	88.23	74.56	39.60	kips
Vni max1 = K1*f'c*Acv =	640.51	518.77	372.86	0.00	kips
Vni max2 = K2*Acv =	640.51	518.77	372.86	0.00	kips
Vni (controlling) =	107.57	88.23	74.56	0.00	kips
Fact. Interface Shear Resistance, Vri=phiVni =	96.81	79.41	67.10	0.00	kips
Fact. Interface Shear Load, Vui = Vu =	12.58	8.87	4.44	0.00	kips
Vu < Vri ?	**OK**	**OK**	**OK**	**N/A**	
Min Interface Shear Reinf, Avf = 0.05*Acv/Fy =	0.667	0.540	0.388	0.000	in^2
Avf > Avfmin ?	**OK**	**OK**	**OK**	**N/A**	

<-- Assumed No Backwall - IGNORE Section 4

5.8.4.3—Cohesion and Friction Factors

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	EWK
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Structure:	Wall Design	**Date:**	October 3, 2014

Stem Reinforcement Cont.

Reinforcement : 3.4

CHECK SHEAR RESISTANCE	SECT 1	SECT 2	SECT 3	SECT 4	AASHTO 5.8.2, 5.8.3.3, 5.8.3.4.2
Factored Shear Force, Vu =	12.58	8.87	4.44	0.00	kips
Factored Moment, Mu =	1794.11	0.00	0.00	0.00	k*in
Resistance Factor, phi: Φ =	0.90	0.90	0.90	0.90	AASHTO 5.5.4.2
Effective Depth: dv =	63.70	51.18	35.98	0.00	in=max((ds-a/2),max(0.9ds,0.72h)) AASHTO 5.8.2.9
H of Stem, h:	66.72	54.04	38.84	0.00	in
Area of Conc on Tension Side, Ac = h*bw/2 =	400.32	324.23	233.04	0.00	in
As (flexural, back face) provd =	0.88	0.66	0.66	0.66	in^2
Max. Size of Coarse Aggregate, ag =	1.50	1.50	1.50	1.50	in
Mu min = Vu*dv =	801.06	454.11	159.60	0.00	k*in
Mu (controlling) =	1794.11	454.11	159.60	0.00	k*in
sx = dv	63.70	51.18	35.98	0.00	in —> See Figure 5.8.3.4.2-3 (Case a)
Crack spg parameter, sxe = 1.38*sx/(ag+0.63) =	41.27	33.16	23.31	0.00	
Strain = ε_s=(Mu/d+Vu)/(Es*As) =	0.0016	0.0009	0.0005	#DIV/0!	
Θ = 29+35000*ε_s =	162.04	94.11	47.05	#DIV/0!	
β = 4.8/(1+750εs)*(51/(39+s_{xe})) =	1.39	2.00	2.92	#DIV/0!	
Nom Shear Resistance, Vn1 =	764.38	614.14	431.75	0.00	kips, Vn = 0.25*f'c*bv*dv AASHTO 5.8.3.3-2
Nominal Shear Resistance: Vn2 = Vc =	67.05	77.67	79.55	#DIV/0!	kips, 0.0316*β*f'c$^{.5}$*bv*dv AASHTO 5.8.3.3-3
Nom Shear Resistance, Vn = min (Vn1, Vn2) =	67.05	77.67	79.55	#DIV/0!	kips
phi*Vn =	60.34	69.90	71.59	#DIV/0!	
Shear OK?	OK	OK	OK	N/A	
Front Face (Dowels) As provd. =	0.44	0.44	0.44	0.44	in^2
As min crack = 0.003*b*sx =	2.29	1.84	1.30	0.00	in^2 —> Only Applicable for Figure 5.8.3.4.2-3 Case B
min (As front, back) > As min ?	N/A	N/A	N/A	N/A	

<-- Assumed No Backwall - IGNORE Section 4

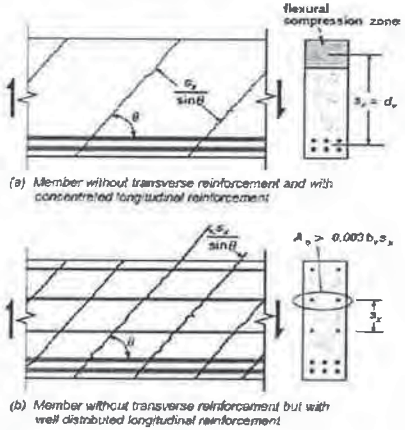


Figure 5.8.3.4.2-3—Definition of Crack Spacing Parameter, s_x

Crack Spacing Parameter, Sx --> Case = Case A

CANTILEVER WALL DESIGN -REINFORCEMENT

General Information

Project Number: 1298\127-1298-12001-LT0077
Description: Khost Bridge No. 10
Structure: Wall Design

Designed By: EWK
Checked By: SAM
Date: October 3, 2014

Results Summary:

Reinforcement : 4.0

REINFORCEMENT RESULTS:

= As Provided / As Required

		STIRUP #	BAR #	SPAC.	REINF. RATIO	FLEX OK?	LBS./ L.F.	LENGTH OF BAR	No. Bars per ft	Wt. of bar PER L.F.	As/LF	SHEAR OK?	
A	TOE(bot):	---	6.00	6.00	5.20	OK	87.32	14.58	2.00	2.99	0.88	OK	
B	HEEL(top):	---	8.00	4.00	1.67	OK	352.74	14.58	3.00	8.06	2.37	OK	
C	STEM 1 (at top of ftg):	0.00	6.00	6.00	1.27	OK	41.92	7.00	2.00	2.99	0.88	OK	
D	STEM 2 (at top of ftg dwl - backface):	0.00	6.00	8.00	1.54	OK	42.42	12.59	1.50	2.25	0.66	OK	
E	STEM 3 (midpt back face):	0.00	6.00	8.00	4.36	OK	42.42	12.59	1.50	2.25	0.66	OK	
F	STEM 4 (at bot of bw):	0.00	6.00	8.00	-0.20	N/A	42.42	12.59	1.50	2.25	0.66	N/A	<-- N/A No Backwall
G	STEM 5 (front face):	0.00	6.00	12.00	---	---	34.11	22.78	1.00	1.50	0.44		
H	STEM 6 (front face dowels):	0.00	6.00	12.00	---	---	3.49	2.33	1.00	1.50	0.44		
I	FOOTING (TOP):	0.00	6.00	12.00	---	---	22.58	1.00	15.08	1.50	0.44		
J	FOOTING (BOT.)	0.00	6.00	12.00	---	---	22.58	1.00	15.08	1.50	0.44		
K	STEM (longitudinal):	0.00	6.00	12.00	---	---	71.21	1.00	47.56	1.50	0.44		
					TOTAL WT. STEEL/FT OF ABUT.=		763.22	LBS./LF					

Section 5
Bill of Quantities

Gardez-Khost Road Project

Section 2 --- Km 27+000 to 65+000

Project Name **Gardez - Khost Road Phase IV - Construction of Bridge #10**

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION		Bill of Quantity (BOQ)-Bridge09			Quantity	Price	
	BoQ Ref. #	**Description**	**UNIT**	**Civil**	**Structural**	**Section 2 Phase IV Construction of Bridge#09**	**Unit Price**	**Total Price**
Division 150 - Project Requirements					-			$0.00
Section 151	**Mobilization**				-			$0.00
	08-151-01	Mobilization	LS		-			$0.00
Section 159	**Demining**				-			$0.00
	08-159-01	De-mining and Technical survey	m2		-			$0.00
	08-159-02	Mine Clearance	m^2		-			$0.00
Section 160	**Snow Removal**				-			$0.00
	08- 160-01	Snow Removal	day		-			$0.00
		Emergency Work	day		-			$0.00
					-			$0.00
					-			$0.00
Division 200 - Earthwork					-			$0.00
Section 201 (Section 31 10 00)	**Clearing and Grubbing**				-			$0.00
	08-201-01	Clearing and Grubbing	ha		-			$0.00
Section 203 (Section 02 41 19)	**Removal of Structure and Obstructions**				-			$0.00
	08-203-01	Removal and disposal of existing structure (Retaining wall, Head wall, wing wall, culverts, lined Ditch)	m^3	374.00	-	374.00		$0.00
	08-203-02	Removal and disposal of existing pavement (asphalt)	m^3	106.00	-	106.00		$0.00
	08-203-03	Removal and Disposal of Existing PCC Pavement	m^3		-			$0.00
	08-203-04	Removal and Disposal of Existing Bridge	each		-	-		$0.00
					-			$0.00
Section 204 (Section 31 20 00, 31 23 19, 31 25 00, & 31 52 13)	**Excavation and Embankment**				-			$0.00
	08- 204-01	Roadway Excavation	m^3	1,584.00	-	1,584.00		$0.00
	08- 204-02	Bridge Excavation	m^3		8,300.00	8,300.00		$0.00
	08- 204-02	River Training Soil Excavation	m^3		-			$0.00
	08- 204-03	Select Topping	m^3		-	-		$0.00
	08- 204-04	Structural Backfill	m^3		10,200.00	10,200.00		$0.00
	08- 204-05	Embankment	m^3	5,935.00	-	5,935.00		$0.00
	08-204-06	Embankment(Granular material with 0-8% passing 75µm sieve)	m^3		-			$0.00
	08-204-07	Erosion Control	m	483.00				$0.00
	08-204-08	Cofferdam (Control/Diversion of Water)	LS		1.00			$0.00

Gardez-Khost Road Project

Section 2 --- Km 27+000 to 65+000

Project Name **Gardez - Khost Road Phase IV - Construction of Bridge #10**

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION		Bill of Quantity (BOQ)-Bridge09			Quantity	Price	
	BoQ Ref. #	Description	UNIT	Civil	Structural	Section 2 Phase IV Construction of Bridge#09	Unit Price	Total Price
Section 205	**Rock Blasting**				-			$0.00
(Section 31 20 00)	08-205-01	Rock Excavation (Inclusive of blasting assumed 10% of total bridge excavtion)	m^3		450.00	450.00		$0.00
					-			$0.00
Division 250- Slope Reinforcement and Retaining wall					-			$0.00
					-			$0.00
Section 251 (Section 31 37 00)	**Riprap**				-			$0.00
	08-251-01	Placed Riprap	m^3		270.00	270.00		$0.00
	08-251-02	Grouted Riprap	m^3	517.00	-	517.00		$0.00
Section 253	**Gabions and Revet Mattresses**				-			$0.00
	08-253-01	Gabions and Revet Mattresses	m^3		-			$0.00
					-			$0.00
Division 300- Aggregate Course					-			$0.00
					-			$0.00
Section 301 (Section 32 12 16)	**Untreated Aggregate Course**				-			$0.00
	08- 301-01	Crushed Aggregate Base Grad. Des. D, 200 mm Carriageway	m^3	398.00	-	398.00		$0.00
	08- 301-02	Crushed Aggregate Base Grad. Des. D, 325 mm, Shoulder	m^3	277.00	-	277.00		$0.00
	08-301-03	Crushed Aggregate Base Grad. Des. D, 300 mm, Side Road	m^3	66.00				
	08- 301-05	Crushed Aggregrate for Underdrain and Under Approach Slab	m^3	17.00	-			$0.00
					-			$0.00
Division 400- Asphalt pavement and surface Treatment					-			$0.00
					-			$0.00
Section 400.3.1	**Asphalt Concrete Surface (Wearing Course)**				-			$0.00
(Section 32 12 16)	08-400.3.1-01	50 mm Asphalt Concrete Surface (Wearing Course)	m^2	2,371.00	-	2,371.00		$0.00
Section 400.3.2	**Asphalt Concrete Binder Course**				-	-		$0.00
(Section 32 12 16)	08-400.3.2-01	75 mm Asphalt Concrete Binder Course	m^2	1,988.00	-	1,988.00		$0.00
Section 411	**Asphalt Prime Coat**				-	-		$0.00
(Section 32 12 16)	08-411-01	Asphalt Prime Coat	m^2	2,621.00	-	2,621.00		$0.00
Section 412	**Asphalt Tack Coat**				-	-		$0.00
(Section 32 12 16)	08-412-01	Asphalt Tack Coat Emulsified Asphalt	m^2	1,988.00	-	1,988.00		$0.00
					-			$0.00

Gardez-Khost Road Project

Section 2 --- Km 27+000 to 65+000

Project Name **Gardez - Khost Road Phase IV - Construction of Bridge #10**

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION		Bill of Quantity (BOQ)-Bridge09			Quantity	Price	
	BoQ Ref. #	Description	UNIT	Civil	Structural	Section 2 Phase IV Construction of Bridge#09	Unit Price	Total Price
Division 500- Rigid Pavement					-			$0.00
					-			$0.00
Section 500.1	**Rigid Pavements**				-			$0.00
	08-501-01	Portland Cement Pavement, 250mm thick (New and patching)	m^2		-			$0.00
					-			$0.00
Division 550-Bridges and Culverts Construction					-			$0.00
Section 552 (Section 03 30 00 & 07 95 66)	**Structural Concrete**				-			$0.00
	08-552-01	Plain Cement Concrete, Class B (15MPa) below footings	m^3		40.00	40.00		$0.00
	08-552-02	Structural Concrete, Class A (25MPa) for reinforced concrete box culverts, cut-off walls, wing walls, sleeper slabs	m^3		-			$0.00
	08-552-03	Plain Cement Concrete Class B (15MPa) below pier and abutment pile caps and approach slabs	m^3		-			$0.00
	08-552-04	Structural Concrete (27.5MPa) for piers,abutments, Walls, and Approach slabs	m^3	2.00	1,200.00	1,202.00		$0.00
	08-552-05	Structural Concrete (27.5MPa) for reinforced concrete deck slabs , beams and diaphragms	m^3		300.00	300.00		$0.00
	08-552-06	Structural Concrete (27.5MPa) for curbs, barriers and sidewalks	m^3		50.00	50.00		$0.00
	08-552-07	Structural Concrete (27.5MPa) for scour mattress	m^3		225.00	225.00		$0.00
	08-552-08	Scuppers	each		4.00	4.00		$0.00
	08-552-09	Weep Holes in Abutments and Walls	each	275.00	12.00	287.00		$0.00
	08-552-10	Strip Seal Joint System	lm		25.00	25.00		$0.00
	08-552-12	PVC Drain Pipe	lm	30.00		30.00		$0.00
Section 554 (Section 03 30 00)	**Reinforcing Steel**				-			$0.00
	08-554-01	Reinforcing steel Grade 60 in abutments, piers, walls and approach slabs	ton		85.00	85.00		$0.00
	08-554-02	Reinforcing steel Grade 60 for Barriers, Curb and Sidewalks	ton		5.00	5.00		$0.00
	08-554-03	Reinforcing steel grade 60 in diaphragms, beams, deck slabs	ton		30.00	30.00		$0.00
	08-554-04	Reinforcing steel grade 60 in scour mattress	ton		25.00	25.00		$0.00

Gardez-Khost Road Project

Section 2 --- Km 27+000 to 65+000

Project Name **Gardez - Khost Road Phase IV - Construction of Bridge #10**

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION		Bill of Quantity (BOQ)-Bridge09			Quantity	Price	
	BoQ Ref. #	Description	UNIT	Civil	Structural	Section 2 Phase IV Construction of Bridge#09	Unit Price	Total Price
Section 556	**Bridge Railing**				-			$0.00
	08-556-01	Concrete Barrier as Bridge Railing, 30 Mpa Structural Concrete	lm		-			$0.00
	08-556-02	Bridge Steel Railing with RC Post	lm		-			$0.00
					-			
Section 559 (Section 07 11 13 & 07 15 53)	**Waterproofing**				-			$0.00
	08-559-01	Waterproofing Membrane	m^2		415.00	415.00		$0.00
	08-559-02	Bituminous Dampproofing	m^2		390.00	390.00		
					-			
Section 564 (Section 07 95 63)	**Bearing Devices**				-			$0.00
	08-564-01	Reinforced Elastomeric Bearings	ea		24.00	24.00		$0.00
					-			$0.00
Section 567	**Subsurface Exploration**				-			$0.00
	08-567-01	Soil Investigation Borings	lm		-			$0.00
	08-567-02	Standard Penetration Testing	tests		-			
	08-567-03	Rock Coring	lm		-			$0.00
	08-567-04	Axial Compressive Testing of Rock Core Samples	each		-			$0.00
	08-567-05	Split Spoon Samples	each		-			$0.00
	08-567-06	Consolidation Test	each		-			$0.00
					-			$0.00
Section 568	**Repair of Bridge Structures**				-			$0.00
	08-568-01	Sealing of Cracks by injection of Epoxy Resin, Conform to AASHTO M 235	m^2		-			$0.00
	08-568-02	Patching of Cracks using Non Shrink Grout, Conform to ASTM C1107	m^2		-			$0.00
					-			$0.00

Gardez-Khost Road Project

Section 2 --- Km 27+000 to 65+000

Project Name **Gardez - Khost Road Phase IV - Construction of Bridge #10**

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION		Bill of Quantity (BOQ)-Bridge09			Quantity	Price	
	BoQ Ref. #	Description	UNIT	Civil	Structural	Section 2 Phase IV Construction of Bridge#09	Unit Price	Total Price
Division 600-Incidental Construction					-			$0.00
Section 602	**Reinforced Concrete Culverts, mortared joints**				-			$0.00
	08- 602-01	RC_Pipe, Ø 610 mm	lm		-			$0.00
	08- 602-02	RC_Pipe, Ø 1000 mm	lm	20.00	-			$0.00
	08- 602-03	RC_Box, 2000x2000 mm	lm		-			$0.00
					-			$0.00
					-			$0.00
Section 607	**Cleaning & Repairing**				-			$0.00
	08-607-03	Cleaning, Reconditioning and Repairing of existing Drainage structure	lm		-			$0.00
Section 608	**Paved Waterways**				-			$0.00
	08-608-01	Type 2_ Class "B" Stone Masonry Lined Ditch A (Trapezoidal)	lm		-			$0.00
Section 620 (Section 32 32 40)	**Stone Masonry**				-			$0.00
	08-620-01	Class "B" _ Retaining Wall, Guardwall, Culvert-Inlet/Outlet Structure, Bed Protection, causeways	m^3	260.00	-	260.00		$0.00
					-			$0.00
Section 633	**Permanent Traffic Control**				-			$0.00
(Section 10 14 01)	08-633-01	Road Signs, Series R/W/I/S, with aluminum panels, retro reflective sheeting type IX, type L-1 letters, galvanized steel posts	ea	5.00	-			$0.00
Section 634 (Section 32 12 16)	**Permanent Pavement Markings**				-			$0.00
	08-634-01	Type "A" Pavement Marking	m^2	126.00	-	126.00		$0.00
Section 638	**Project Information Signages**				-			$0.00
	08-638-01	Project Information Signages	ls		-			$0.00
					-			$0.00
		Total Estimated Cost						

Bill of Quantities Back-up

Gardez-Khost Road Project

Section 2 --- Km 27+000 to 65+000

Project Name — Gardez - Khost Road Phase IV - Construction of Bridge #10

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION BoQ Ref. #	Description	UNIT	Civil
Division 150 - Project Requirements				
Section 151	**Mobilization**			
	08-151-01	Mobilization	LS	
Section 159	**Demining**			
	08-159-01	De-mining and Technical survey	m^2	
	08-159-02	Mine Clearance	m^2	
Section 160	**Snow Removal**			
	08- 160-01	Snow Removal	day	
		Emergency Work	day	
Division 200 - Earthwork				
Section 201 (Section 31 10 00)	**Clearing and Grubbing**			
	08-201-01	Clearing and Grubbing	ha	
Section 203	**Removal of Structure and Obstructions**			
	08-203-01	Removal and disposal of existing structure (Retaining wall, Head wall, wing wall, culverts, lined Ditch)	m^3	374.00
	08-203-02	Removal and disposal of existing pavement (asphalt)	m^3	106.00
	08-203-03	Removal and Disposal of Existing PCC Pavement	m^3	
	08-203-04	Removal and Disposal of Existing Bridge	each	
Section 204	**Excavation and Embankment**			
	08- 204-01	Roadway Excavation	m^3	1584.00
	08- 204-02	Bridge Excavation	m^3	
	08- 204-02	River Training Soil Excavation	m^3	
	08- 204-03	Select Topping	m^3	
	08- 204-04	Structural Backfill	m^3	
	08- 204-05	Embankment	m^3	5935.00
	08-204-06	Embankment(Granular material with 0-8% passing 75µm sieve)	m^3	
	08-204-07	Erosion Control	m	483.00
Section 205	**Rock Blasting**			
(Section 31 20 00)	08-205-01	Rock Excavation (Inclusive of blasting assumed 10% of total bridge excavtion)	m^3	
Division 250- Slope Reinforcement and Retaining wall				
Section 251	**Riprap**			
	08-251-01	Placed Riprap (at Abutments & Piers 1,500mm thick)	m^3	
	08-251-02	Grouted Riprap	m^3	517.00
Section 253	**Gabions and Revet Mattresses**			
	08-253-01	Gabions and Revet Mattresses	m^3	
Division 300- Aggregate Course				
Section 301	**Untreated Aggregate Course**			
	08- 301-01	Crushed Aggregate Base Grad. Des. D, 200 mm Carriageway	m^3	398.00
	08- 301-02	Crushed Aggregate Base Grad. Des. D, 325 mm, Shoulder	m^3	277.00
	08-301-03	Crushed Aggregate Base Grad. Des. D, 300 mm, Access Road	m^3	66.00
	08- 301-05	Crushed Aggregrate for Underdrain and Under Approach Slab	m^3	17.00
Division 400- Asphalt pavement and surface Treatment				
Section 400.3.1	**Asphalt Concrete Surface (Wearing Course)**			
(Section 32 12 16)	08-400.3.1-01	50 mm Asphalt Concrete Surface (Wearing Course)	m^2	2371.00

USAID Spec No.	ITEM DESCRIPTION			
(UFGS Spec No.)	BoQ Ref. #	Description	UNIT	Civil
Section 400.3.2	Asphalt Concrete Binder Course			
(Section 32 12 16)	08-400.3.2-01	75 mm Asphalt Concrete Binder Course	m^2	1988.00
Section 411	Asphalt Prime Coat			
(Section 32 12 16)	08-411-01	Asphalt Prime Coat	m^2	2621.00
Section 412	Asphalt Tack Coat			
(Section 32 12 16)	08-412-01	Asphalt Tack Coat Emulsified Asphalt	m^2	1988.00
Division 500- Rigid Pavement				
Section 500.1	Rigid Pavements			
	08-501-01	Portland Cement Pavement, 250mm thick (New and patching)	m^2	
Division 550-Bridges and Culverts Construction				
Section 552	Structural Concrete			
	08-552-01	Plain Cement Concrete, Class B (15MPa) below footings	m^3	
	08-552-02	Structural Concrete, Class A (25MPa) for reinforced concrete box culverts, cut-off walls, wing walls, sleeper slabs	m^3	
	08-552-03	Plain Cement Concrete Class B (15MPa) below pier and abutment pile caps and approach slabs	m^3	
	08-552-04	Structural Concrete (27.5MPa) for piers,abutments, Walls, and Approach slabs	m^3	2.00
	08-552-05	Structural Concrete (27.5MPa) for reinforced concrete deck slabs ,beams and diaphragms	m^3	
	08-552-06	Structural Concrete (27.5MPa) for curbs, barriers and sidewalks	m^3	
	08-552-09	Drainage Spouts in Super-structure	each	
	08-552-10	Weep Holes in Abutments and Walls	each	275.00
	08-552-11	Asphaltic Bridge Joints	lm	
	08-552-12	PVC Drain Pipe	lm	30.00
Section 554	Reinforcing Steel			
	08-554-01	Reinforcing steel Grade 60 for abutments, piers, walls and approach slabs	ton	
	08-554-02	Reinforcing steel Grade 60 for Barriers, Curb and Sidewalks	ton	
	08-554-03	Reinforcing steel grade 60 for diaphragms, beams, deck slabs	ton	
Section 556	Bridge Railing			
	08-556-01	Concrete Barrier as Bridge Railing, 30 Mpa Structural Concrete	lm	
	08-556-02	Bridge Steel Railing with RC Post	lm	
Section 559	Waterproofing			
	08-559-01	Waterproofing Membrane	m^2	
	08-559-02	Bituminous Dampproofing	m^2	
Section 564	Bearing Devices			
	08-564-01	Reinforced Elastomeric Bearings	lm	
Section 567	Subsurface Exploration			
	08-567-01	Soil Investigation Borings	lm	
	08-567-02	Standard Penetration Testing	tests	
	08-567-03	Rock Coring	lm	
	08-567-04	Axial Compressive Testing of Rock Core Samples	each	
	08-567-05	Split Spoon Samples	each	
	08-567-06	Consolidation Test	each	
Section 568	Repair of Bridge Structures			
	08-568-01	Sealing of Cracks by injection of Epoxy Resin, Conform to AASHTO M 235	m^2	

USAID Spec No.	ITEM DESCRIPTION			
(UFGS Spec No.)	BoQ Ref. #	Description	UNIT	Civil
	08-568-02	Patching of Cracks using Non Shrink Grout, Conform to ASTM C1107	m^2	
Division 600-Incidental Construction				
Section 602	**Reinforced Concrete Culverts, mortared joints**			
	08- 602-01	RC_Pipe, Ø 610 mm	lm	
	08- 602-02	RC_Pipe, Ø 1000 mm	lm	20.00
	08- 602-03	RC_Box, 2000x2000 mm	lm	
Section 607	**Cleaning & Repairing**			
	08-607-03	Cleaning, Reconditioning and Repairing of existing Drainage structure	lm	
Section 608	**Paved Waterways**			
	08-608-01	Type 2_ Class "B" Stone Masonry Lined Ditch A (Trapezoidal)	lm	
Section 620	**Stone Masonry**			
	08-620-01	Class "B" _ Retaining Wall, Guardwall, Culvert-Inlet/Outlet Structure, Bed Protection, causeways	m^3	260.00
Section 633	**Permanent Traffic Control**			
	08-633-01	Road Signs, Series R/W/I/S, with aluminum panels, retro reflective sheeting type IX, type L-1 letters, galvanized steel posts	ea	5.00
Section 634	**Permanent Pavement Markings**			
	08-634-01	Type "A" Pavement Marking	m^2	126.00
Section 638	**Project Information Signages**			
	08-638-01	Project Information Signages	ls	
		Total Estimated Cost		

ITEM No. 08-203-01 - REMOAL AND DISPOSAL OF EXISTING STRUCTURE **CU.M.**

Station	Description	Depth (m)	Area (sq.m.)	Volume (cu.m.)
151+830 - 151+867.53 LT	Retaining Wall	2.00	47.96	95.9
151+867.53 - 151+959.52 LT	Slope Protection	0.30	215.67	64.7
151+867.53 - 151+964.86 LT	Guard Wall	1.60	60.13	96.2
151+929.24 - 151+959.19	Retaining Wall	3.00	18.23	54.7
152+005.88	Culvert and Headwalls			43.8

Subtotal	355.3	cu.m.
Contingency (5%)	17.8	cu.m.
Total for Item	373.1	cu.m.

SAY **374** **CU.M.**

TETRA TECH
One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 1 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-203-02 - REMOAL AND DISPOSAL OF EXISTING PAVEMENT **CU.M.**

Station	Description	Area (sq.m.)	Thickness (m)	Volume (cu.m.)
151+800.00 - 151+882.17	Pavement Limits	638.55	0.125	79.8
152+076.97 - 152+080.00	Pavement Limits	164.64	0.125	20.6

Subtotal	100.4	cu.m.
Contingency (5%)	5	cu.m.
Total for Item	105.4	cu.m.

SAY **106** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 2 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-204-01 - Roadway Excavation **CU.M.**

(See Attached Cut-Fill Estimate)

Subtotal	1508.0	cu.m.
Contingency (5%)	75.4	cu.m.
Total for Item	1583.4	cu.m.

SAY **1584** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10

SHEET NO. 3 OF 21

CALCULATED BY: ANF DATE: 10/12/2014

CHECKED BY: JKM DATE:

ITEM No. 08-204-05 - Embankment **CU.M.**

(See Attached Cut-Fill Estimate)

Subtotal	4945.70	cu.m.
Contingency (20%)	989.1	cu.m.
Total for Item	5934.8	cu.m.

SAY 5935 CU.M.

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10

CALCULATED BY: ANF DATE: 10/12/2014

CHECKED BY: JKM DATE:

ITEM No. 08-204-07 - EROSION CONTROL **LM**

Station	Description	Length (l.m.)
151+812.09 - 151+945.42 RT	Erosion Control North of Bridge RT	133.33
151+944 RT	Erosion Control Along Access Road A	73.09
151+951.58 - 151+962.46 RT	Erosion Control North of Bridge RT	10.88
151+997.15 - 152+012.49 RT	Erosion Control South of Bridge RT	14.95
152+020 RT	Erosion Control Along Access Road B	20.25
152+027.27 - 152+100 RT	Erosion Control South of Bridge RT	72.73
151+843 - 151+963.7 LT	Erosion Control North of Bridge LT	120.70
151+997.19 - 152+010.58 LT	Erosion Control South of Bridge LT	13.39

Subtotal	459.32	LM
Contingency (5%)	22.97	LM
Total for Item	482.29	LM

SAY **483** **LM**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 5 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-251-02 - GROUTED RIP RAP **CU.M.**

Station	Description	Area (sq.m.)	Width (m)	Volume (cu.m.)
151+843 - 151+963.7 LT	Slope Protection LT			311.80
151+996.3 - 152+055 LT	Slope Protection LT			100.16
151+953.21 - 151+967.81 RT	Slope Protection RT			28.43
151+992.19 - 152+017.75 RT	Slope Protection RT			51.57

Subtotal	491.95	cu.m.
Contingency (5%)	24.6	cu.m.
Total for Item	516.55	cu.m.

SAY **517** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 6 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-301-01 - CRUSHED AGGREGATE BASE GRAD. DES, 200mm CARRIAGEWAY **CU.M.**

Station	Description	Area (sq.m.)	Thickness (m)	Volume (cu.m.)
151+800.00 - 151+957.20	Pavement Limits	1164.056	0.200	232.81
152+002.80 -152+100	Pavement Limits	729.051	0.200	145.81

(Areas from AutoCAD)
(Including paved portion of access road)

Subtotal	378.62	cu.m.
Contingency (5%)	18.93	cu.m.
Total for Item	397.55	cu.m.

SAY **398** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 7 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-301-02 - CRUSHED AGGREGATE BASE GRAD. DES, 325mm SHOULDER **CU.M.**

Station	Description	Area (sq.m.)	Thickness (m)	Volume (cu.m.)
151+800 - 151+962.46 LT	Shoulder Limits North of Bridge			80.7
151+997.54 - 152+100 LT	Shoulder Limits South of Bridge			55.3
151+800 - 151+962.46 RT	Shoulder Limits North of Bridge			79.4
151+997.54 - 152+100 RT	Shoulder Limits South of Bridge			47.6

(Calculated from cross-sections)

Subtotal	263	cu.m.
Contingency (5%)	13.2	cu.m.
Total for Item	276.2	cu.m.

SAY **277** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 8 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-301-03 - CRUSHED AGGREGATE BASE GRAD. DES, 300mm ACCESS ROADS **CU.M.**

Station	Description			Volume (cu.m.)
151+944 RT	Access Road A			53.11
152+020 RT	Access Road B			9.21

Subtotal	62.32	cu.m.
Contingency (5%)	3.1	cu.m.
Total for Item	65.42	cu.m.

SAY 66 CU.M.

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 9 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-301-05 - CRUSHED AGGREGATE FOR UNDERDRAIN AND UNDER APPROACH SLAB CU.M.

Station	Description			Volume (cu.m.)
151+962.00	Underdrain			2.04
151+998.00	Underdrain			2.04
151+957.20 - 151+962.2	Approach Slab			6
151+997.80 - 152+002.80	Approach Slab			6

(Calculated from Cross Sections)

Subtotal	16.08	cu.m.
Contingency (5%)	0.8	cu.m.
Total for Item	16.88	cu.m.

SAY 17 CU.M.

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10

SHEET NO. 10 OF 21

CALCULATED BY: ANF DATE: 10/12/2014

CHECKED BY: JKM DATE:

ITEM No. 08-400.3.1-01 - 50mm ASPHALT CONCRETE SURFACE (WEARING COUSRE) **SQ.M.**

Station	Description			Area (sq.m.)
151+800 - 152+100	Pavement Limits			2257.917

(goes over bridge, carriageway only)
(Including paved portion of access road)
(Area from AutoCAD)

Subtotal	2257.917	sq.m.
Contingency (5%)	112.9	sq.m.
Total for Item	2370.817	sq.m.

SAY **2371** **SQ.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 11 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-400.3.2-01 - 75mm ASPHALT CONCRETE BINDER COURSE **SQ.M.**

Station	Description			Area (sq.m.)
151+800.00 - 151+957.20	Pavement Limits			1164.056
152+002.80 -152+100	Pavement Limits			729.051

(does not go over bridge, carriageway only)
(Including paved portion of access road)
(Not over approach slab)
(Areas from AutoCAD)

Subtotal	1893.107	sq.m.
Contingency (5%)	94.7	sq.m.
Total for Item	1987.807	sq.m.

SAY **1988** **SQ.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 12 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-411-01 - ASPHALT PRIME COAT **SQ.M.**

Station	Description			Area (sq.m.)
151+800.00 - 151+957.20	Pavement Limits			1547.482
152+002.80 -152+100	Pavement Limits			948.247

(does not go over bridge, carriageway + shoulder)	Subtotal	2495.729	sq.m.
(Including paved portion of access road)	Contingency (5%)	124.8	sq.m.
(Areas from AutoCAD)	Total for Item	2620.529	sq.m.

SAY **2621** **SQ.M.**

TETRA TECH
One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 13 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-412-01 - 75mm ASPHALT TACK COAT EMULSIFIED ASPHALT **SQ.M.**

Station	Description			Area (sq.m.)
151+800.00 - 151+957.20	Pavement Limits			1164.056
152+002.80 -152+100	Pavement Limits			729.051

(does not go over bridge, carriageway only)
(Including paved portion of access road)
(Not over approach slab)
(Areas from AutoCAD)

Subtotal	1893.107	sq.m.
Contingency (5%)	94.7	sq.m.
Total for Item	1987.807	sq.m.

SAY **1988** **SQ.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 14 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-552-04 STRUCTURAL CONCRETE (27.5 Mpa) **CU.M.**

Description	Length (m)	Width (m)	Height (m)	Quantity	Volume (cu.m.)
Signs Posts	0.7	0.3	0.3	5	0.3
Curbs	2	0.250	0.45	4	0.9

Subtotal	1.2	cu.m.
Contingency (5%)	0.1	cu.m.
Total for Item	1.3	cu.m.

SAY **2** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 15 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-552-10 Weep Holes in Grouted Riprap Slope Protection **EA**

Station	Description			Quantity (EA)
151+843 - 151+963.7 LT	Slope Protection LT			170
151+996.3 - 152+055 LT	Slope Protection LT			40
151+953.21 - 151+967.81 RT	Slope Protection RT			24
151+992.19 - 152+017.75 RT	Slope Protection RT			28

Subtotal	262	EA
Contingency (5%)	13	EA
Total for Item	275	EA

SAY **275** **EA**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10

SHEET NO. 16 OF 21

CALCULATED BY: ANF DATE: 10/12/2014

CHECKED BY: JKM DATE:

ITEM No. 08-552-12 PVC Drain Pipe **L.M.**

Station	Description			Length (l.m.)
151+962.00	Drain Pipe			14
151+998.00	Drain Pipe			14

Subtotal	28	l.m.
Contingency (5%)	1.4	l.m.
Total for Item	29.4	l.m.

SAY **30** **L.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 17 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-602-02 RC_Pipe, Ø 1000 mm **L.M.**

Station	Description				Length (l.m.)
151+947.55 - 151+957.46 RT	2-1000mm RCPC				14.22
151+953 - 151+962.46 RT	1-1000mm RCPC				4.5

Subtotal	18.72	l.m.
Contingency (5%)	0.9	l.m.
Total for Item	19.62	l.m.

SAY **20** **L.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 18 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

ITEM No. 08-620-01 STONE MASONRY **CU.M.**

Station	Description	Length (m)	Width (m)	Height (m)	Volume (cu.m.)
151+830 - 151+962.46 LT	Guardwall	135.19	0.5	1.625	109.8
151+953 - 151+962.46 RT	Guardwall	9.75	0.5	1.625	7.9
151+997.54 - 152+055 LT	Guardwall	56.72	0.5	1.625	46.1
151+997.54 - 152+010 RT	Guardwall	12.46	0.5	1.625	10.1
151+943.98 - 151+958.07 RT	Culvert				41.68
152+014.72 - 152+025.18 RT	Culvert				31.58

Subtotal	247.16	cu.m.
Contingency (5%)	12.4	cu.m.
Total for Item	259.56	cu.m.

SAY **260** **CU.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10

SHEET NO. 19 OF 21

CALCULATED BY: ANF DATE: 10/12/2014

CHECKED BY: JKM DATE:

ITEM No. 08-633-01 Signs **EA**

Station	Description	Quantity (Each)
151+860 RT	DW-6hd	1
Access Road (151+950.92 RT)	RI-I	1
Access Road (152+021.62 RT)	RI-I	1
152+040 LT	DW-1	1
152+095 LT	DW-6G2	1

Total:	5	EA

SAY **5** **EA**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10

SHEET NO. 20 OF 21

CALCULATED BY: ANF DATE: 10/12/2014

CHECKED BY: JKM DATE:

ITEM No. 08-634-01- TYPE "A" PAVEMENT MARKING **SQ.M.**

Station	Description	Length (m)	Width (m)	Area (Sq.m.)
151+800.00 - 152+100.00	Double Centerline	300	0.200	60
151+800.00 - 151+939.90	Right Edgeline: Solid	139.9	0.100	13.99
151+939.9 - 151+956.43	Right Edgeline: Dashed	16.53	0.100	1.653
151+956.43 - 152+012.49	Right Edgeline: Solid	56.06	0.100	5.606
152+012.49 - 152+027.27	Right Edgeline: Dashed	14.78	0.100	1.478
152+027.27 - 152+100	Right Edgeline: Solid	72.73	0.100	7.273
151+800.00 - 152+100.00	Left Edgeline: Solid	300	0.100	30

Subtotal	120	sq.m.
Contingency (5%)	6	sq.m.
Total for Item	126	sq.m.

SAY **126** **SQ.M.**

TETRA TECH

One Grant Street
Framingham, MA 01703-9005
(508) 903-2000

JOB NO. 127-1298-12001-LT0077-Bridge No. 10
SHEET NO. 21 OF 21
CALCULATED BY: ANF DATE: 10/12/2014
CHECKED BY: JKM DATE:

CUT FILL ESTIMATIONS

	CUT							FILL						
STATION	X-SEC AREA LEFT (m^2)	X-SEC AREA RIGHT (m^2)	AVG. AREA LEFT (m^2)	AVG. AREA RIGHT (m^2)	CUT QUANTITY LEFT (m^3)	CUT QUANTITY RIGHT (m^3)	CUT QUANTITY TOTAL (m^3)	X-SEC AREA LEFT (m^2)	X-SEC AREA RIGHT (m^2)	AVG. AREA LEFT (m^2)	AVG. AREA RIGHT (m^2)	FILL QUANTITY LEFT (m^3)	FILL QUANTITY RIGHT (m^3)	FILL QUANTITY TOTAL (m^3)
151+800	1.461	1.461						0.000	0.000					
			1.006	0.958	20.110	19.160	39.270			0.000	2.808	0.000	56.150	56.150
151+820	0.550	0.455						0.000	5.615					
			0.275	0.228	5.500	4.550	10.050			2.601	6.112	52.010	122.240	174.250
151+840	0.000	0.000						5.201	6.609					
			0.000	0.000	0.000	0.000	0.000			7.041	7.647	140.820	152.930	293.750
151+860	0.000	0.000						8.881	8.684					
			0.000	0.035	0.000	0.690	0.690			9.687	9.503	193.740	190.060	383.800
151+880	0.000	0.069						10.493	10.322					
			0.000	0.035	0.000	0.690	0.690			12.985	10.987	259.700	219.740	479.440
151+900		0.000						15.477	11.652					
			0.000	0.000	0.000	0.000	0.000			15.756	13.428	315.110	268.550	583.660
151+920	0.000	0.000						16.034	15.203					
			0.000	0.000	0.000	0.000	0.000			16.397	15.295	327.940	305.890	633.830
151+940	0.000	0.000						16.760	15.386					
			0.000	0.000	0.000	0.000	0.000			17.105	15.572	342.100	311.430	653.530
151+960	0.000	0.000						17.450	15.757					
152+000	0.000	0.000						10.335	9.767					
			0.000	0.000	0.000	0.000	0.000			9.114	14.993	182.270	299.860	482.130
152+020	0.000	0.000						7.892	20.219					
			0.000	0.000	0.000	0.000	0.000			6.745	15.045	134.890	300.900	435.790
152+040	0.000	0.000						5.597	9.871					
			0.295	0.000	5.890	0.000	5.890			3.168	6.494	63.350	129.880	193.230
152+060	0.589	0.000						0.738	3.117					
			0.568	0.000	2.272	0.000	2.272			0.430	2.695	1.718	10.778	12.496
152+064	0.547	0.000						0.121	2.272					
			0.990	0.433	15.832	6.920	22.752			0.061	1.136	0.968	18.176	19.144
152+080	1.432	0.865						0.000	0.000					
			1.692	1.446	33.830	28.910	62.740			0.000	0.000	0.000	0.000	0.000
152+100	1.951	2.026						0.000	0.000					
Cut/Fill for River Channel East and West of Bridge							1,363.7							600.66
Subtract Crushed Aggregate Base for Access Roads from Total Fill														-56.2
				TOTAL	**83.4**	**60.9**	**1,508.0**				**TOTAL**	**2,014.6**	**2,386.6**	**4,945.7**

Gardez-Khost Road Project

Section 2 — Km 27+000 to 65+000

Project Name — Gardez - Khost Road Phase IV - Construction of Bridge #10

USAID Spec No. (UFGS Spec No.)	ITEM DESCRIPTION BoQ Ref. #	Description	UNIT	Structural
Division 150 - Project Requirements				
Section 151	**Mobilization**			
	08-151-01	Mobilization	LS	
Section 159	**Demining**			
	08-159-01	De-mining and Technical survey	m^2	
	08-159-02	Mine Clearance	m^2	
Section 160	**Snow Removal**			
	08- 160-01	Snow Removal	day	
		Emergency Work	day	
Division 200 - Earthwork				
Section 201 (Section 31 10 00)	**Clearing and Grubbing**			
	08-201-01	Clearing and Grubbing	ha	
Section 203 (Section 02 41 19)	**Removal of Structure and Obstructions**			
	08-203-01	Removal and disposal of existing structure (Retaining wall, Head wall, wing wall, culverts, lined Ditch)	m^3	
	08-203-02	Removal and disposal of existing pavement (asphalt)	m^3	
	08-203-03	Removal and Disposal of Existing PCC Pavement	m^3	
	08-203-04	Removal and Disposal of Existing Bridge	each	1.00
Section 204 (Section 31 20 00, 31 23 19, 31 25 00, & 31 52 13)	**Excavation and Embankment**			
	08- 204-01	Roadway Excavation	m^3	
	08- 204-02	Bridge Excavation	m^3	8,300.00
	08- 204-02	River Training Soil Excavation	m^3	
	08- 204-03	Select Topping	m^3	
	08- 204-04	Structural Backfill	m^3	10,200.00
	08- 204-05	Embankment	m^3	
	08-204-06	Embankment(Granular material with 0-8% passing 75μm sieve)	m^3	
	08-204-07	Erosion Control	m	
	08-204-08	Cofferdam (Control/Diversion of Water)	LS	1.00
Section 205	**Rock Blasting**			
(Section 31 20 00)	08-205-01	Rock Excavation (Inclusive of blasting assumed 10% of total bridge excavtion)	m^3	450.00
Division 250- Slope Reinforcement and Retaining wall				
Section 251 (Section 31 37 00)	**Riprap**			
	08-251-01	Placed Riprap	m^3	270.00
	08-251-02	Grouted Riprap	m^3	
Section 253	**Gabions and Revet Mattresses**			
	08-253-01	Gabions and Revet Mattresses	m^3	
Division 300- Aggregate Course				
Section 301 (Section 32 12 16)	**Untreated Aggregate Course**			
	08- 301-01	Crushed Aggregate Base Grad. Des. D, 200 mm Carriageway	m3	
	08- 301-02	Crushed Aggregate Base Grad. Des. D, 325 mm, Shoulder	m3	
	08-301-03	Crushed Aggregate Base Grad. Des. D, 300 mm, Side Road	m3	
	08- 301-05	Stone Aggregate for Catch Trench, 75 mm (max.)	m3	
Division 400- Asphalt pavement and surface Treatment				
Section 400.3.1	**Asphalt Concrete Surface (Wearing Course)**			
(Section 32 12 16)	08-400.3.1-01	50 mm Asphalt Concrete Surface (Wearing Course)	m^2	

Section 400.3.2	**Asphalt Concrete Binder Course**			
(Section 32 12 16)	08-400.3.2-01	75 mm Asphalt Concrete Binder Course	m^2	
Section 411	**Asphalt Prime Coat**			
(Section 32 12 16)	08-411-01	Asphalt Prime Coat	m^2	
Section 412	**Asphalt Tack Coat**			
(Section 32 12 16)	08-412-01	Asphalt Tack Coat Emulsified Asphalt	m^2	
Division 500- Rigid Pavement				
Section 500.1	**Rigid Pavements**			
	08-501-01	Portland Cement Pavement, 250mm thick (New and patching)	m^2	
Division 550-Bridges and Culverts Construction				
Section 552	**Structural Concrete**			
(Section 03 30 00 & 07 95 65)	08-552-01	Plain Cement Concrete, Class B (15MPa) below footings	m^3	40.00
	08-552-02	Structural Concrete, Class A (25MPa) for reinforced concrete box culverts, cut-off walls, wing walls, sleeper slabs	m^3	
	08-552-03	Plain Cement Concrete Class B (15MPa) below pier and abutment pile caps and approach slabs	m^3	
	08-552-04	Structural Concrete (27.5MPa) for piers,abutments, Walls, and Approach slabs	m^3	1,200.00
	08-552-05	Structural Concrete (27.5MPa) for reinforced concrete deck slabs ,beams and diaphragms	m^3	300.00
	08-552-06	Structural Concrete (27.5MPa) for curbs, barriers and sidewalks	m^3	50.00
	08-552-07	Structural Concrete (27.5MPa) for scour mattress	m^3	225.00
	08-552-08	Scuppers	each	4.00
	08-552-09	Weep Holes in Abutments and Walls	each	12.00
	08-552-10	Strip Seal Joint System	lm	25.00
Section 554	**Reinforcing Steel**			
(Section 03 30 00)	08-554-01	Reinforcing steel Grade 60 for abutments, piers, walls and approach slabs	ton	85.00
	08-554-02	Reinforcing steel Grade 60 for Barriers, Curb and Sidewalks	ton	5.00
	08-554-03	Reinforcing steel grade 60 for diaphragms, beams, deck slabs	ton	30.00
	08-554-04	Reinforcing steel grade 60 in scour mattress	ton	25.00
Section 556	**Bridge Railing**			
	08-556-01	Concrete Barrier as Bridge Railing, 30 Mpa Structural Concrete	lm	
	08-556-02	Bridge Steel Railing with RC Post	lm	
Section 559	**Waterproofing**			
(Section 07 11 13 & 07 15 53)	08-559-01	Waterproofing Membrane	m^2	415.00
	08-559-02	Bituminous Dampproofing	m^2	390.00
Section 564	**Bearing Devices**			
(Section 07 95 63)	08-564-01	Reinforced Elastomeric Bearings	ea	24.00
Section 567	**Subsurface Exploration**			
	08-567-01	Soil Investigation Borings	lm	
	08-567-02	Standard Penetration Testing	tests	
	08-567-03	Rock Coring	lm	
	08-567-04	Axial Compressive Testing of Rock Core Samples	each	
	08-567-05	Split Spoon Samples	each	
	08-567-06	Consolidation Test	each	
Section 568	**Repair of Bridge Structures**			
	08-568-01	Sealing of Cracks by injection of Epoxy Resin, Conform to AASHTO M 235	m^2	
	08-568-02	Patching of Cracks using Non Shrink Grout, Conform to ASTM C1107	m^2	

Division 600-Incidental Construction				
Section 602	**Reinforced Concrete Culverts, mortared joints**			
	08- 602-01	RC_Pipe, Ø 610 mm	lm	
	08- 602-02	RC_Pipe, Ø 1000 mm	lm	
	08- 602-03	RC_Box, 2000x2000 mm	lm	
Section 607	**Cleaning & Repairing**			
	08-607-03	Cleaning, Reconditioning and Repairing of existing Drainage structure	lm	
Section 608	**Paved Waterways**			
	08-608-01	Type 2._ Class "B" Stone Masonry Lined Ditch A (Trapezoidal)	lm	
Section 620 (Section 32 32 40)	**Stone Masonry**			
	08-620-01	Class "B" _ Retaining Wall, Guardwall, Culvert-Inlet/Outlet Structure, Bed Protection, causeways	m^3	
Section 633	**Permanent Traffic Control**			
	08-633-01	Road Signs, Series R/W/I/S, with aluminum panels, retro reflective sheeting type IX, type L-1 letters, galvanized steel posts	ea	
Section 634 (Section 32 12 16)	**Permanent Pavement Markings**			
	08-634-01	Type "A" Pavement Marking	m^2	
Section 638	**Project Information Signages**			
	08-638-01	Project Information Signages	ls	
		Total Estimated Cost		

Reinforced Concrete Beams Quantities

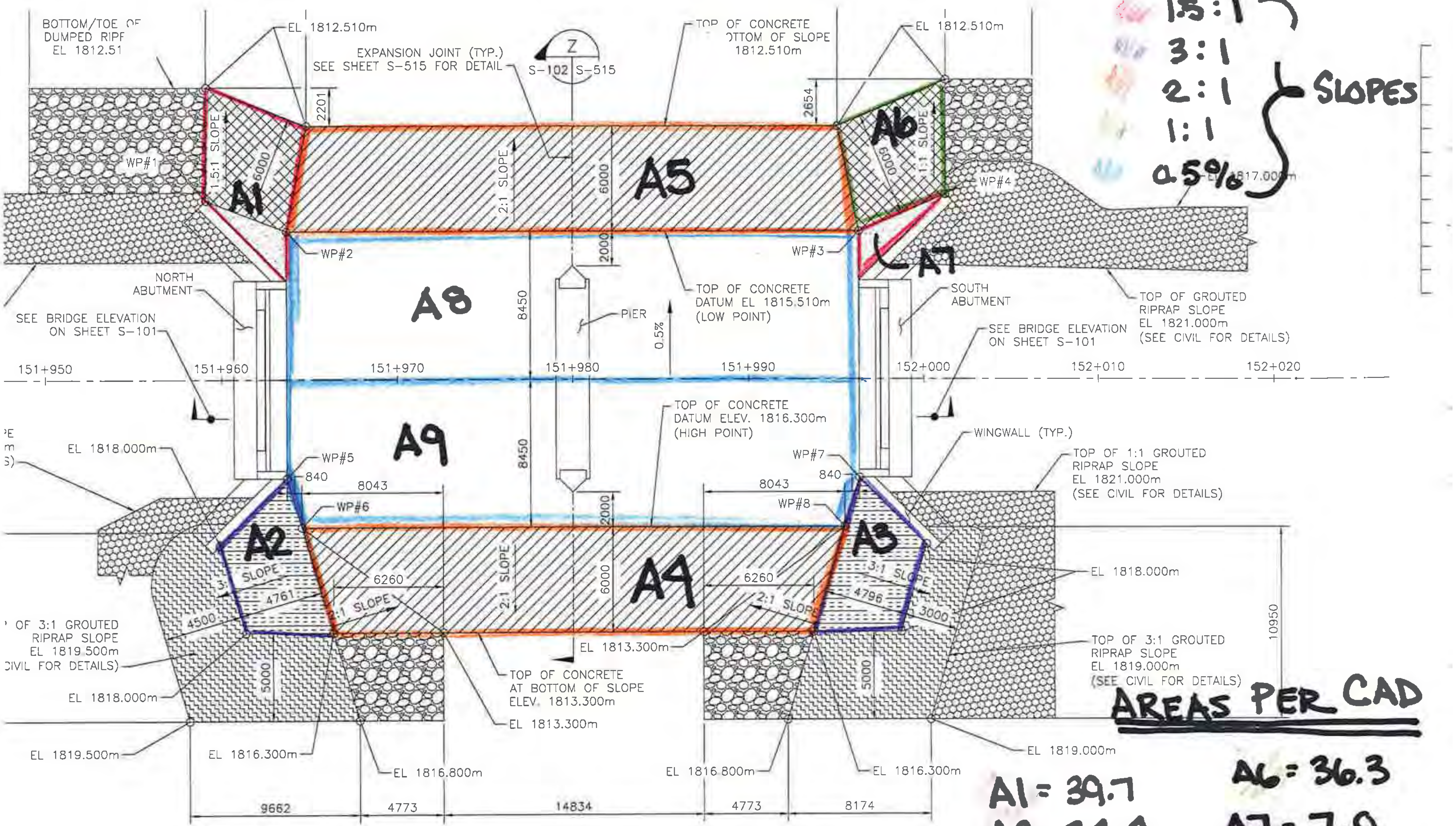
Loading HL93

Bridge Length =	33.6	m
Span Length =	16.8	m
No. of Spans =	2	

Concrete	Length (m)	Width (m)	Height (m)	Volume (m^3)	Qty	Total Volume (m^3)	
Abutments							
Abutment Footing	13.45	7.00	1.50	141.23	2	282.45	
Abutment Stem - North	11.25	1.70	5.96	113.99	1	113.99	
Abutment Stem - South	11.25	1.70	5.79	110.77	1	110.77	
Abutment Backwall	10.35	0.75	1.55	12.03	2	24.06	
Abutment Batter - North	11.25	0.65	5.06	37.00	1	37.00	
Abutment Batter - South	11.25	0.65	4.89	35.77	1	35.77	
Abutment Cheek Wall	1.70	0.45	1.50	1.15	4	4.59	
Approach Slab	5.00	11.25	0.30	16.88	2	33.75	
Approach Slab Support	11.25	0.62		6.98	2	13.95	<--- Area taken from CAD
Abutment Totals						656.33	
Piers							
Pier Footing	13.45	5.60	1.50	112.98	1	112.98	
Pier Stem	11.40	1.50	4.20	71.84	1	71.84	
Pier Ends	0.75	0.75	4.20	2.36	2	4.73	
Pier Cap	11.40	1.90	0.68	14.62	1	14.62	
Pier Keeper	0.53	1.90	1.50	1.50	2	2.99	
Pier Totals						207.16	
Superstructure							
Beam	16.80	0.60	1.50	15.12	12	181.44	
End Diaphragm	1.25	0.73	1.28	1.16	10	11.55	
Pier Diaphragm	1.25	0.73	1.28	1.16	10	11.55	
Interior Diaphagm	1.25	0.30	1.25	0.47	10	4.69	
Deck	33.60	10.95	0.23	82.78	1	82.78	= 292.02 CM
Barrier	33.60	0.28	1.40	12.94	2	25.87	
Sidewalk	33.60	1.20	0.27	10.81	2	21.61	= 47.48 CM
Superstructure Totals						339.50	
Walls (Calculated using avg widths and avg heights)							
Wall Stem - Southeast	7.00	1.15	6.18	49.75	1	49.75	
Wall Stem - Northwest	6.00	1.15	6.41	44.22	1	44.22	
Wall Stem - Northeast	6.50	1.15	6.22	46.48	1	46.48	
Wall Stem - Southwest	6.00	1.15	6.28	43.33	1	43.33	
Wall Footing - Southeast	7.00	4.60	1.00	32.20	1	32.20	
Wall Footing - Northwest	6.00	4.60	1.00	27.60	1	27.60	
Wall Footing - Northeast	6.50	4.60	1.00	29.90	1	29.90	
Wall Footing - Southwest	6.00	4.60	1.00	27.60	1	27.60	
Wall Totals						301.08	
Concrete Apron (Scour Pad)							
Concrete Apron (Scour Pad)	(See Attached Calcs)			225.00	1	225.00	
Concrete Apron (Scour Pad) Totals						225.00	
Concrete Totals						1729	
Say —>						1775 m^3	

Plain Concrete Class B (15Mpa) Lean Concrete below Footings	Length m	Width m	Height m	Volume m^3	Qty	Total Volume m^3
Abutment	13.45	7.00	0.10	9.42	2	18.83
Pier	13.45	5.60	0.10	7.53	1	7.53
Southeast Wall	7.00	4.60	0.10	3.22	1	3.22
Northwest	6.00	4.60	0.10	2.76	1	2.76
Northeast	6.50	4.60	0.10	2.99	1	2.99
Southwest	6.00	4.60	0.10	2.76	1	2.76
Total						38.09
Say —>						40 m^3

	Totals (Without Contingency)		
Substructure Concrete Totals	863 Cubic Meter, -->	Say	875
Deck Beams & Diaphgrams Concrete Totals	292 Cubic Meter, -->	Say	300
Barrier & Sidewalks Concrete Totals	47 Cubic Meter, -->	Say	50
Wall Concrete Totals	301 Cubic Meter, -->	Say	325
Piers, Abutments, Walls, Approach Slab Concrete Totals	1165 Cubic Meter, -->	Say	1200
Concrete Apron Totals	225 Cubic Meter, -->	Say	225
Total Concrete	**1729 Cubic Meter, —>**	**Say**	**1775**
Total Plain Concrete	**38 Cubic Meter, —>**	**Say**	**40**



AREAS PER CAD

A1 = 39.7
A2 = 34.4
A3 = 34.5
A4 = 156.0
A5 = 188.6
A6 = 36.3
A7 = 7.0
A8 = 275.5
A9 = 273.1

[ALL AREAS IN M²]

NOTES:

1. ALL ELEVATIONS NOTED ARE ALONG THE TOP OF CONCRETE MATTRESS.
2. EXPANSION JOINTS TO BE PROVIDED IN SCOUR MATTRESS AS INDICATED. CONSTRUCTION JOINTS ARE OPTIONAL, AS REQUIRED FOR CONSTRUCTION. SEE SHEET S-515 FOR JOINT DETAILS.
3. SEE SHEET S-515 FOR CONCRETE MATTRESS REINFORCEMENT DETAILS.
4. THE CONCRETE MATTRESS SHALL BE 200mm THICK.

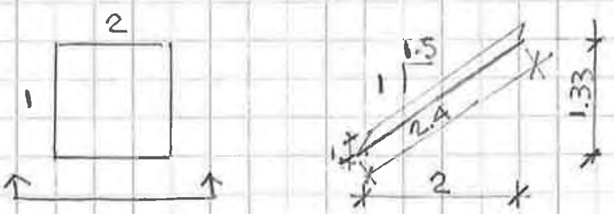
BRIDGE #10

CONCRETE MATTRESS LAYOUT PLAN
SCALE 1:150

SLOPE FACTORS

AREAS A_1 TO A_9 SEE PLAN
*ALL AREAS PER CAD.

= 1.5 : 1



FLAT AREA = (1)(2) = 2 SLOPED AREA = (2.4)(1) = 2.4 ∴ FACTOR = 2X = 2.4 ∴ X = 1.2

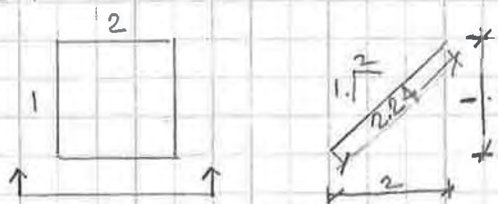
A1 = 39.7 m^2
A7 = 7.0 m^2
46.7 × 1.2 = 56.04 m^2

= 3 : 1

SLOPED AREA = $\frac{\sqrt{2^2 + (.67)^2}\,(1)}{(2)(1)}$ = 1.05

A2 = 34.4 m^2
A3 = 34.5 m^2
68.9 × 1.05 = 72.35 m^2

= 2 : 1



FLAT AREA = (1)(2) = 2 SLOPED AREA = (2.24)(1) = 2.24 ∴ FACTOR = 2X = 2.24 ∴ X = 1.12

A4 = 156 m^2
A5 = 188.6 m^2
344.6 m^2 × 1.12 = 386 m^2

TETRA TECH

One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB BRIDGE 10 - BOQ
SHEET NO. ______ OF ______
CALCULATED BY NAV DATE 10/8/14
CHECKED BY ______ DATE ______
SCALE ______

SLOPE FACTORS [CONT.]

= 1:1

$$\text{FACTOR} = \frac{\sqrt{2^2+2^2}\,(1)}{(2)(1)} = 1.41$$

$A6 = 36.3 m^2 \times 1.41 = 51.18 m^2$

= 0.5% ← PRETTY FLAT ∴ FACTOR = 1

$A8 = 275.5 m^2$
$A9 = 273.1 m^2$
$548.6 m^2$

TOTAL

56.04
72.35
386
51.18
548.6

$1114.17 m^2 \times 0.2 m$ thick

TOTAL VOL: $222.8 m^3$

SAY $225 m^3$

TETRA TECH

One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB BRIDGE 10
SHEET NO. ____ OF ____
CALCULATED BY nav DATE 10/8/14
CHECKED BY ____ DATE ____
SCALE ____

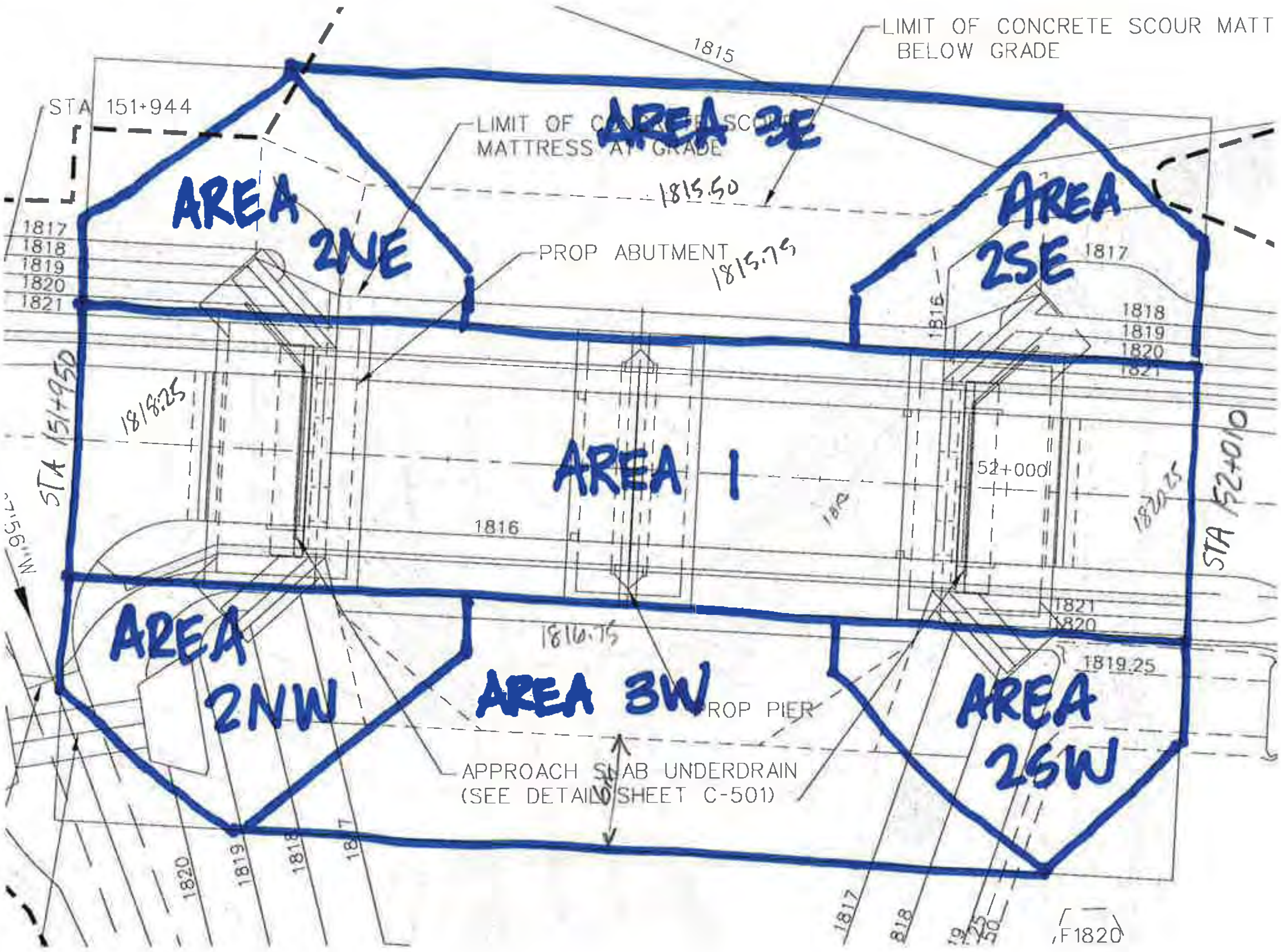
Reinforcement Steel	Volume (m^3)	Volume (cy)	#Steel/ cy Conc	Steel (lbs)	Qty	Total Steel (lbs)			Totals (Without Contingency)	
Abutments										
Abutment Footing	141.23	184.72	120	22165.83	2	44331.66				
Abutment Stem - North	113.99	149.09	100	14908.67	1	14908.67				
Abutment Stem - South	110.77	144.88	100	14488.42	1	14488.42				
Abutment Backwall	12.03	15.74	150	2360.56	2	4721.13				
Abutment Batter - North	37.00	48.40	100	4839.58	1	4839.58				
Abutment Batter - South	35.77	46.79	100	4678.90	1	4678.90				
Abutment Keeper Block	1.15	1.50	150	225.13	4	900.52				
Approach Slab	16.88	22.07	150	3310.75	2	6621.50				
Approach Slab Support	6.98	9.12	150	1368.44	2	2736.89				
Abutment Totals						98227.26	= 44555 kg			
Piers										
Pier Footing	112.98	147.77	120	17732.66	1	17732.66				
Pier Stem	71.84	93.96	100	9395.93	1	9395.93				
Pier Ends	2.36	3.09	150	463.62	2	927.23				
Pier Keeper	1.50	1.96	150	293.55	2	587.11				
Pier Totals						28642.93	= 12992 kg	Substructure Reinforcement Totals	57.5 Metric Tons, -->	Say 60
Superstructure										
Beam	15.12	19.78	150	2966.43	12	35597.17				
End Diaphragm	1.16	1.51	150	226.69	10	2266.94				
Pier Diaphragm	1.16	1.51	150	226.69	10	2266.94				
Interior Diaphagm	0.47	0.61	150	91.97	10	919.65				
Deck	82.78	108.27	200	21654.94	1	21654.94	= 28443 kg	Deck, Beams & Diaphgrams Reinforcement Totals	28.4 Metric Tons, -->	Say 30
Barrier	12.94	16.92	150	2537.95	2	5075.89				
Sidewalk + Curb	10.81	14.13	150	2120.01	2	4240.02	= 4226 kg	Barrier & Sidewalks Reinforcement Totals	4.2 Metric Tons, -->	Say 5
Superstructure Totals						72021.56				
Walls (Calculated using avg widths and avg heights)										
Wall Stem - Southeast	49.75	65.07	100	6506.92	1	6506.92				
Wall Stem - Northwest	44.22	57.83	100	5783.13	1	5783.13				
Wall Stem - Northeast	46.48	60.79	100	6079.29	1	6079.29				
Wall Stem - Southwest	43.33	56.68	100	5667.61	1	5667.61				
Wall Footing - Southeast	32.20	42.12	120	5053.92	1	5053.92				
Wall Footing - Northwest	27.60	36.10	120	4331.93	1	4331.93				
Wall Footing - Northeast	29.90	39.11	120	4692.92	1	4692.92				
Wall Footing - Southwest	27.60	36.10	120	4331.93	1	4331.93				
Wall Totals						42447.65	= 19254 kg	Wall Reinforcement Totals	19.3 Metric Tons, -->	Say 25
								Piers, Abutments, Walls, Approach Slab Reinforcement Totals	76.8 Metric Tons, -->	Say 85
Concrete Apron (Scour Pad)										
Concrete Apron (Scour Pad)	225.00	294.29	175.00	51500.53	1	51500.53				
Concrete Apron (Scour Pad) Totals						51500.53	= 23360 kg	Concrete Apron Totals	23.4 Metric Tons, -->	Say 25
Steel Totals						292839.93	lbs			
Steel Totals						132830	kg			
Say -->						133000	kg			
Steel Totals						133	Metric Tons	**Reinforcement Totals**	**132830 Kg, -->**	**Say 133000**
Say -->						145	Metric Tons	**Reinforcement Totals**	**132.8 Metric Tons, -->**	**Say 145**

Excavation	Length (m)	Width (m)	Top Length (m)	Top Width (m)	Height (m)	Volume (m^3)	Qty	Total Volume (m^3)
Substructure								
Area 1 (Abutement/Pier)	14.65		Area =	262.00		3838.30	1	3838.30
Abutment & Pier Excavation								3838.30
Wall - Southwest		Area =	156.00		3.75	585.00	1	585.00 m^3
Wall - Northwest		Area =	156.00		3.00	468.00	1	468.00
Wall - Northeast		Area =	159.00		2.50	397.50	1	397.50
Wall - Southeast		Area =	162.00		2.75	445.50	1	445.50
Wall Excavation								1896.00
Apron Excavation (North1)	35.00		Area =	22.00		770.00	1	770.00 m^3
Apron Excavation (North2)	11.00		Area =	3.00		33.00	1	33.00
Apron Excavation (South1)	35.00		Area =	30.00		1050.00	1	1050.00
Apron Excavation (South2)	11.00		Area =	5.00		55.00	1	55.00 m^3
Berm Excavation	35.00		Area =	30.00		1050.00	1	1050.00
Apron Excavation								2958.00
Excavation Totals								8692 m^3
Say -->								8750
Soil Excavation Totals								8258 m^3
Say -->								8300
Rock Excavation Totals								435 m^3
Say -->								450

Backfill	Bot Length (m)	Bot Width (m)	Top Length (m)	Top Width (m)	Height (m)	Volume (m^3)	Qty	Total Volume (m^3)
Abutment								
Substructure								
Area 1 (Abutement/Pier)	14.65		Area =	202.00		2959.30	1	2959.30
Abutment & Pier Backfill								2959.30
Wall - Southwest		Area =	156.00		6.60	1029.60	1	1029.60
Wall - Northwest		Area =	156.00		6.60	1029.60	1	1029.60
Wall - Northeast		Area =	159.00		5.10	810.90	1	810.90
Wall - Southeast		Area =	162.00		4.60	745.20	1	745.20
Wall Backfill								3615.30
Apron Excavation (North1)	35.00		Area =	22.00		770.00	1	770.00
Apron Excavation (North2)	11.00		Area =	3.00		33.00	1	33.00
Apron Excavation (South1)	35.00		Area =	30.00		1050.00	1	1050.00
Apron Excavation (South2)	11.00		Area =	5.00		55.00	1	55.00
Apron Backfill								1908.00
Backfill Totals								8482.60
Backfill Total + 20% Swell								10179.12
Say -->								10200
RiprapTotals (See Hand Calcs)								270.00 m^3
Say -->								270

	Totals (Without Contingency)		
Soil Exc. (95%)	3646 Cubic Meter, -->	Say	3650
Rock Exc. (5%)	192 Cubic Meter, -->	Say	200
Soil Exc. (95%)	1801 Cubic Meter, -->	Say	1825
Rock Exc. (5%)	95 Cubic Meter, -->	Say	100
Soil Exc. (95%)	2810 Cubic Meter, -->	Say	2825
Rock Exc. (5%)	148 Cubic Meter, -->	Say	150
Excavation Totals	**8692 Cubic Meter, -->**	**Say**	**8750**
Soil Excavation Totals	**8258 Cubic Meter, -->**	**Say**	**8300**
Rock Excavation Totals	**435 Cubic Meter, -->**	**Say**	**450**
Backfill Abutment	2959 Cubic Meter, -->	Say	2975
Rip Rap Abutment	-2959 Cubic Meter, -->	Say	0
Backfill Wall	3615 Cubic Meter, -->	Say	3625
Rip Rap Abutment	-3615 Cubic Meter, -->	Say	0
Backfill Pier	1908 Cubic Meter, -->	Say	2475
Rip Rap Pier	-1908 Cubic Meter, -->	Say	0
Rip Rap Total	**8483 Cubic Meter, -->**	**Say**	**11344**
Rip Rap Total	**-8483 Cubic Meter, -->**	**Say**	**270**

BRIDGE EXCAVATION

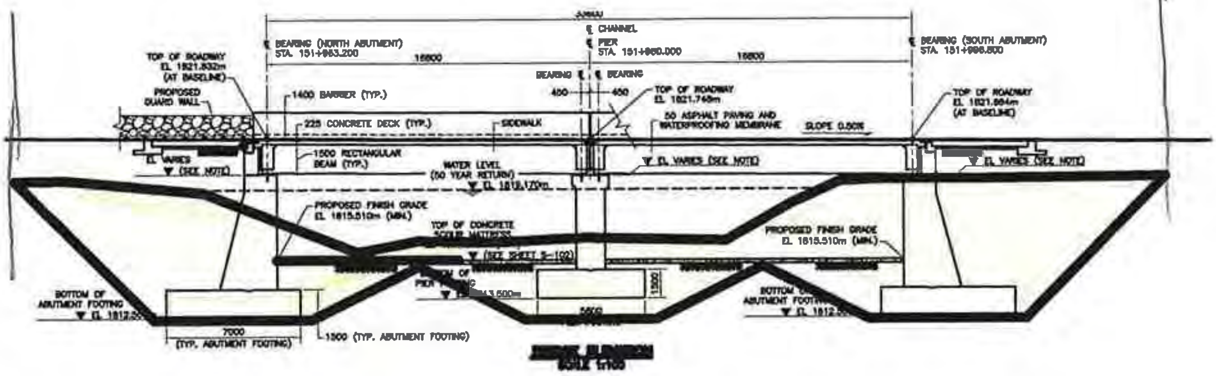


SUMMARY

AREA	VEX (CM)
1	3840
2NE	400
2NW	470
2SE	450
2SW	585
3E	800
3W	2150
TOTAL	8700CM

BRIDGE EXCAVATION – AREA 1

STA 151+950 STA 152+010



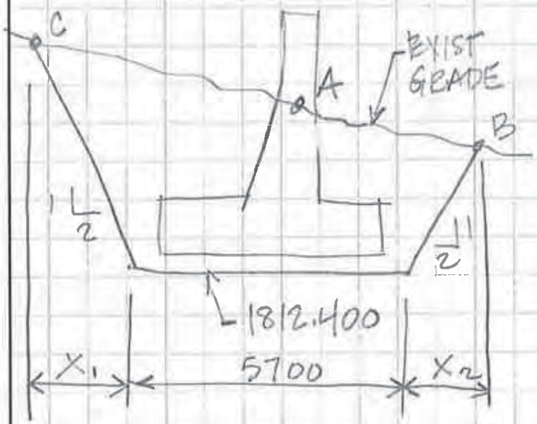
A = 262 m² (measured in CAD)

THIS SECTION CONTINUES ALONG THE CHANNEL A DISTANCE OF 14.65 m.

V = 262 (14.65) = 3840 cm

BRIDGE EXCAVATION - AREA 2NE/2NW/2SE/2SW

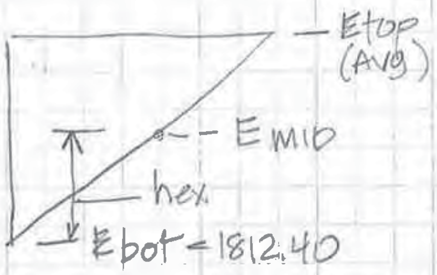
RETAINING WALLS



WALL	ELEVATIONS A	B	C	X1	X2
NE	1816	1815.5	1818.25	6.2	11.5
NW	1818.	→		11	11
SE	1819	1815.75	1820.25	6.5	9.0
SW	1820.5	1820.5	1819	12.0	11

X1 & X2, BASED ON A 2:1 SLOPE ARE OVERLY CONSERVATIVE. CONSIDER X1 = X2 = 6.0m. ALSO CONSIDER THIS DISTANCE AT THE END OF THE WALL

SEE FOLLOWING PAGES FOR IMPACT AREAS FOR EXCAVATION. OVER THESE AREAS, CONSIDER AN AVERAGE HEIGHT FOR QUANTITY PURPOSES.

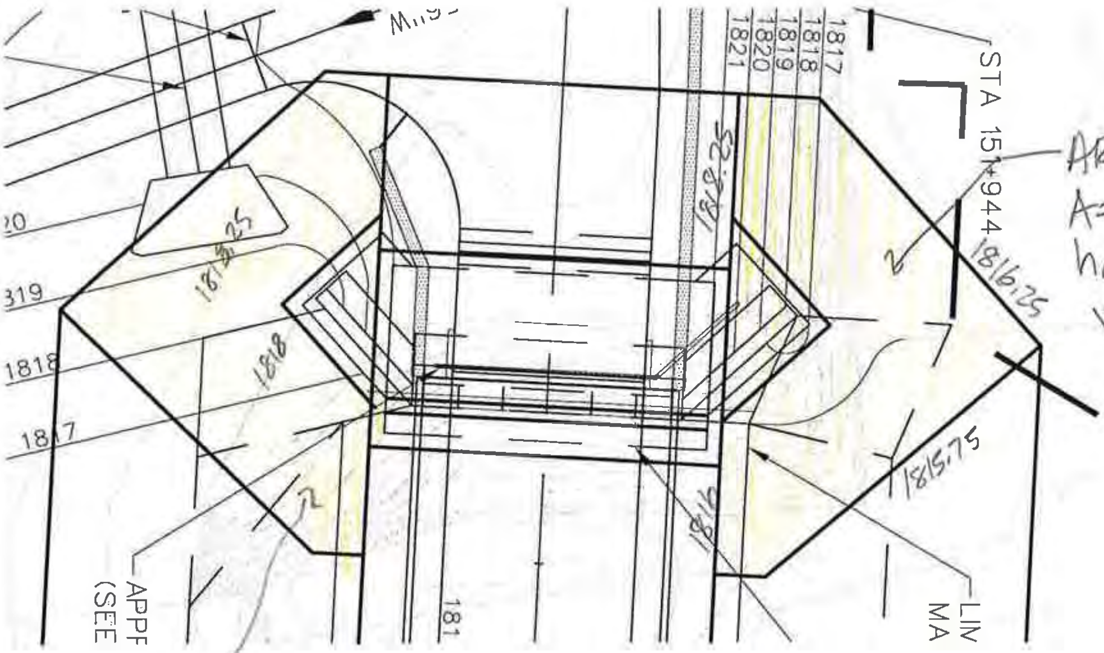


WALL	Etop	hex
NE	1817 +/-	2.5m +/-
NW	1818.25 +/-	3.0m +/-
SE	1818 +/-	2.75m +/-
SW	1820 +/-	3.75m +/-

TETRA TECH

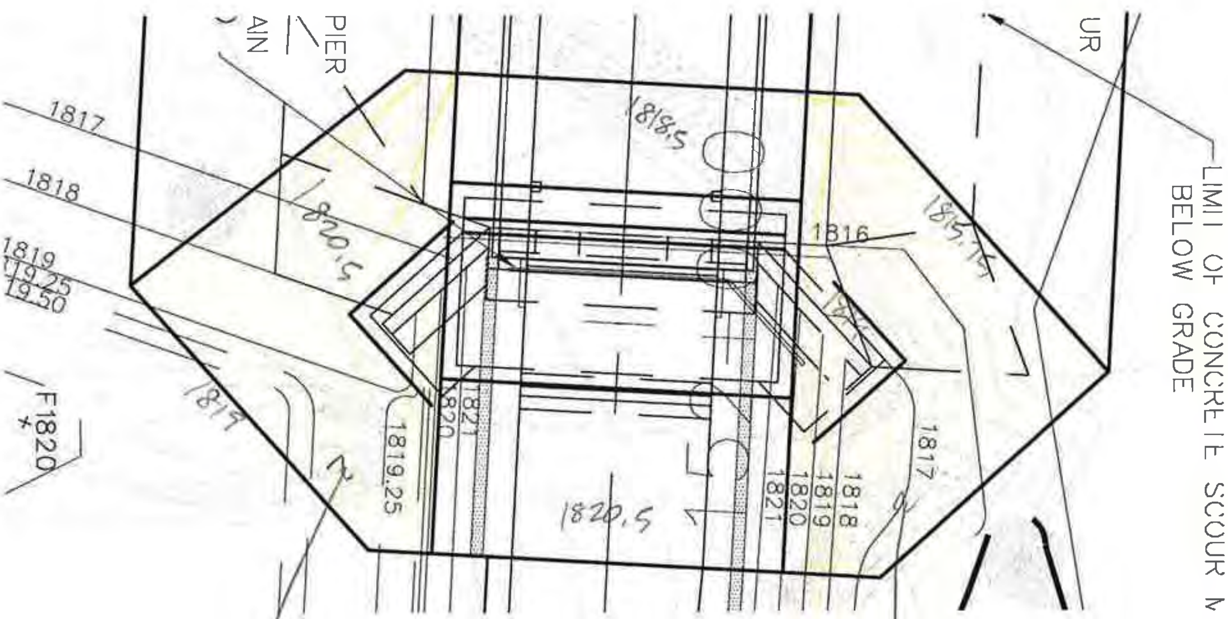
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB: BRIDGE 10
SHEET NO.: OF:
CALCULATED BY: APC DATE: 10/2/14
CHECKED BY: DATE:
SCALE:



AREA 2NE
A=159 SM
havg=2.5M
V=400CM

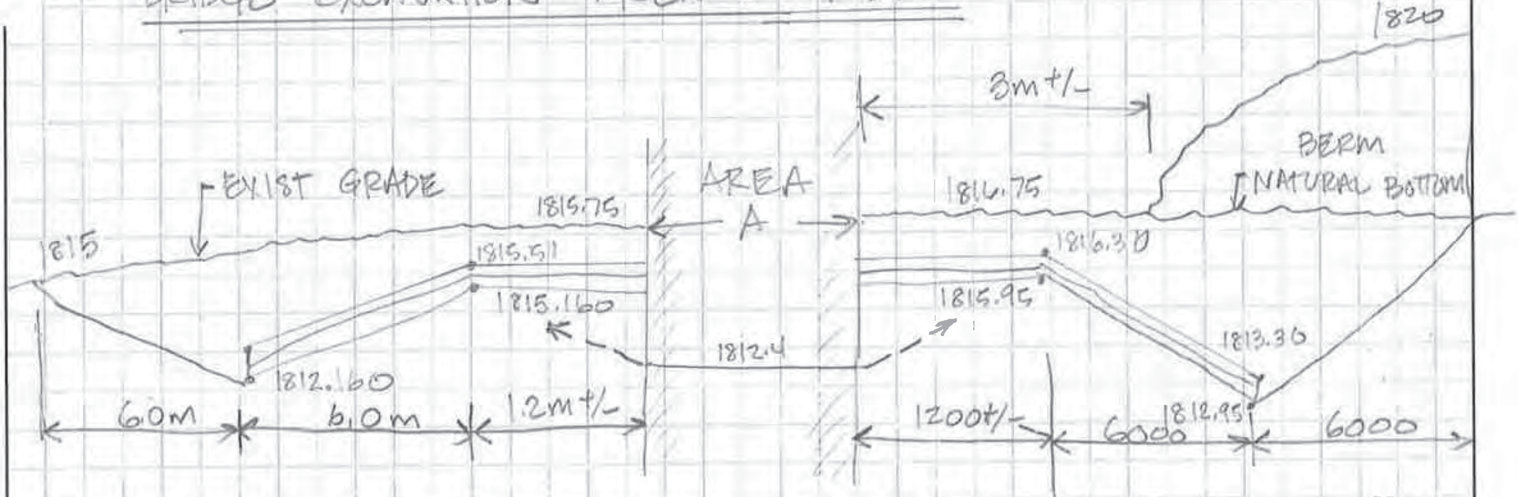
AREA 2NW
A= 156SM
havg = 3.0m
V= 470 CM



AREA 2SW
A= 156SM
havg =3.75m
V= 585CM

AREA 2SE
A= 162 SM
havg =2.75m
V=450cm

BRIDGE EXCAVATION - AREA 3E & 3W



FIRST CONTRACTOR WILL REMOVE BERM

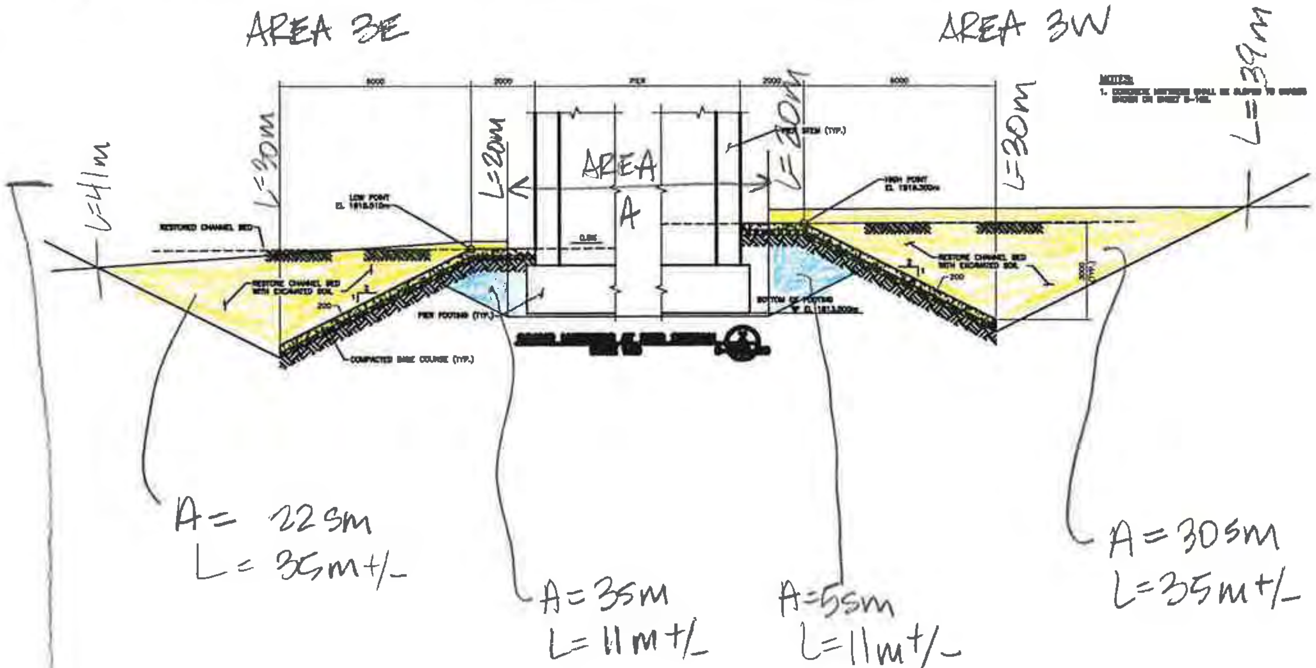
Aberm ≈ 10m x 3m = 30 SM (TO EDGE OF BRIDGE EX AREA)

AREA OF EXCAVATION REQUIRED FOR SCOUR PAD SHOWN.
AREA OF EXCAVATION REQUIRED FOR PIER (CONTINUATION OF AREA A) WILL OVERLAP THE SCOUR PAD EXCAVATION.
SEE FOLLOWING PAGE.

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB BRIDGE 10
SHEET NO. ____ OF ____
CALCULATED BY APL DATE 10/2/14
CHECKED BY ____ DATE ____
SCALE ____

BRIDGE EXCAVATION - AREA 3E & 3W



IN PLAN VIEW, THE LENGTH OF THE SCOUR PAD EXCAVATION VARIES. TO BE CONSERVATIVE, SAY L=35m+/-.

AREA 3E

$V_1 = 22(35) = 770$ CM
$V_2 = 3(11) = 33$ CM

800CM

AREA 3W

$V_1 = 30(35) = 1050$ CM
$V_2 = 5(11) = 55$ CM

1100CM + BERM

BERM = 30 SM x 35m+/- = 1050 cm

TOTAL = 2150 cm

ROCK EXCAVATION

PROVIDE ALLOWANCE FOR POTENTIAL EXCAVATION INVOLVING ROCK.

ASSUME 5% OF EXCAVATION

5% x 8700 = 435 CM

$\boxed{\text{TOTAL} = 435\text{ CM}}$

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SHEET NO. ____ OF ____

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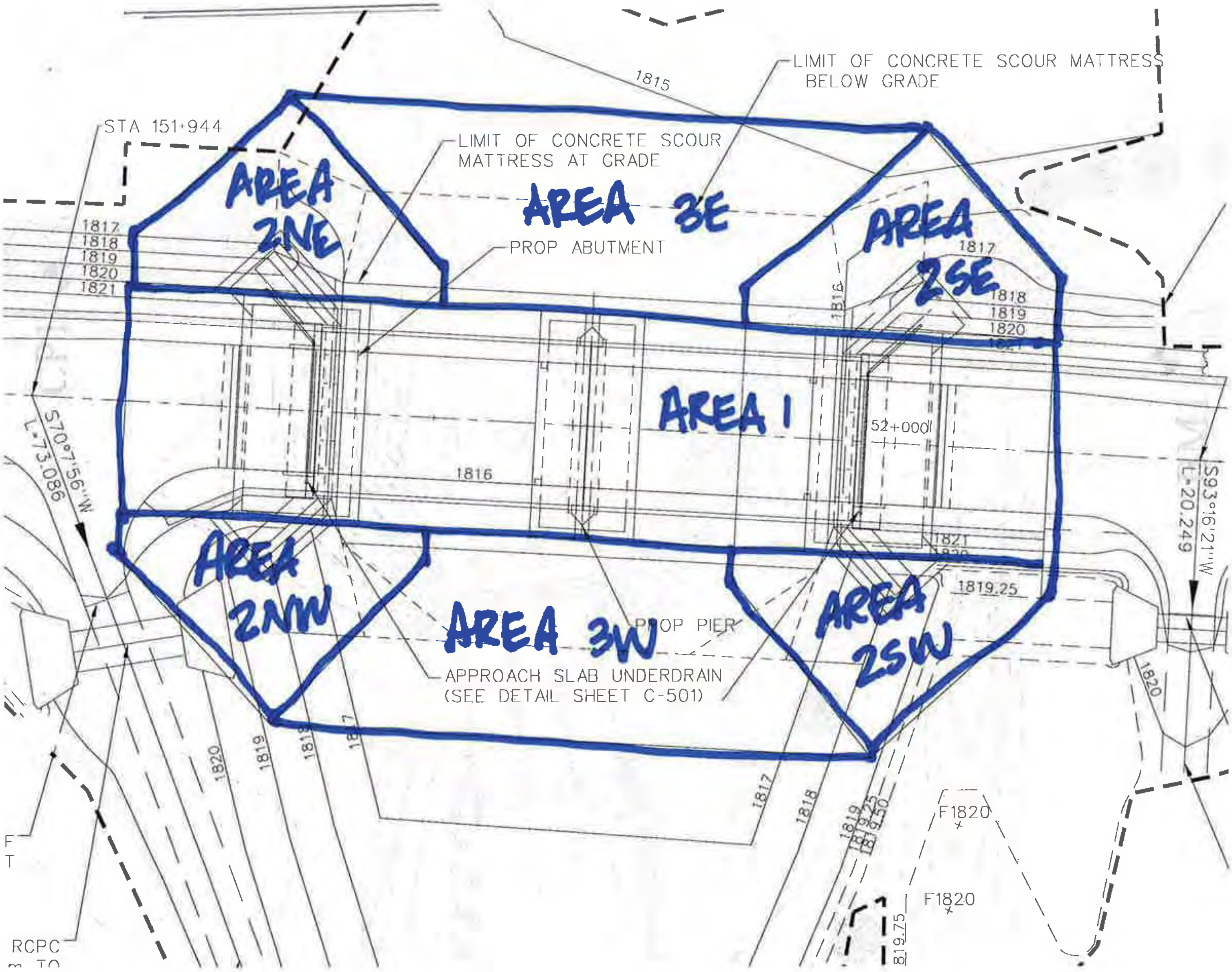
SCALE ____

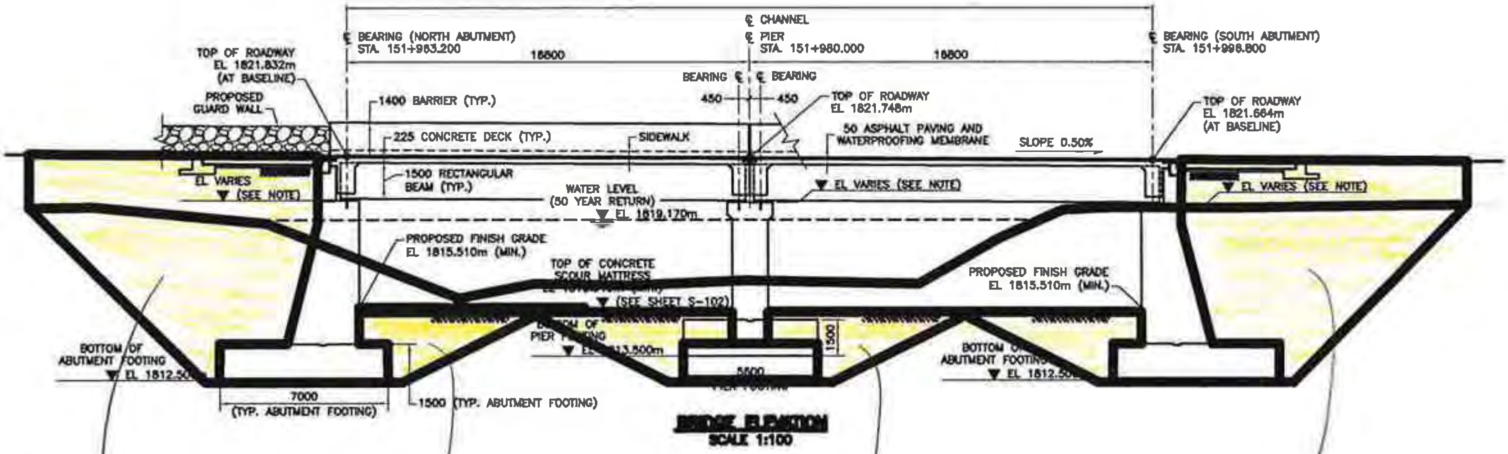
BRIDGE BACKFILL



SUMMARY

AREA	V BACKFILL (CM)
1	2960
2NE	810
2NW	1030
2SE	745
2SW	1030
3E	800
3W	1100
SUBTOTAL	8475 cm
+ 20% SWELL	1700 cm
TOTAL	**10,200 CM**

BRIDGE BACKFILL- AREA 1



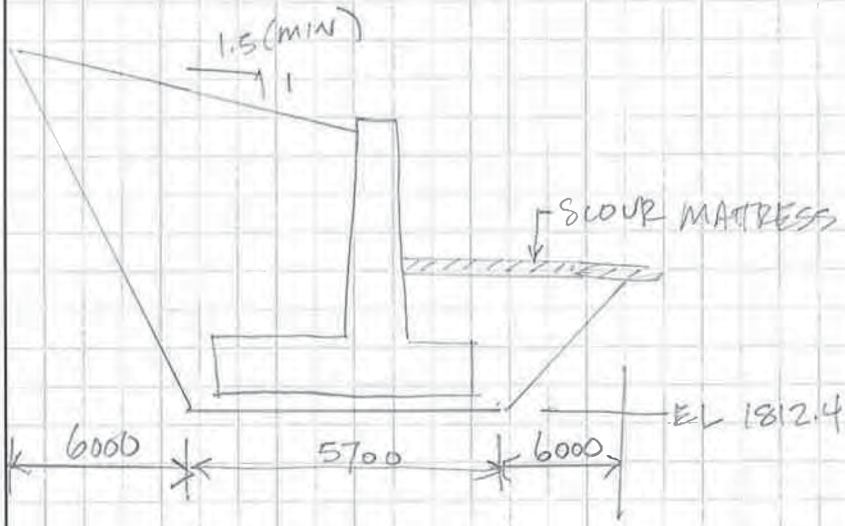
A=78SM A=25SM A=25SM A=74SM

ATOTAL = 202 SM

L= 14.65 M

V= 2960 CM

BACKFILL - AREA 2NE / 2NW / 2SE / 2SW



SIMILAR TO THE EXCAVATION SKETCHES, THE BACKFILL BEHIND THE WALL OVERLAPS IN AREA WITH THE ABUTMENT BACKFILL AND THE BACKFILL IS PREDOMINANTLY IN FRONT OF OR ON THE END OF THE WALL.

CONSIDER AN AVERAGE TOP OF BACKFILL, SIMILAR TO APPROACH FOR EXCAVATION.

WALL	Etop (+/-)	Hfill (+/-)	A (sm)	V (cm)
NE	1817.5	5.1 m	159	810
NW	1819	6.6 m	156	1030
SE	1817	4.6 m	162	745
SW	1819	6.6 m	156	1030

FROM CAD, SEE EXCAVATION CALCS.

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JOB: AESP BRIDGE 10
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CHECKED BY: ______ DATE: ______
SCALE: ______

BACKFILL - AREA 3E & 3W

BACKFILL AREA IS APPROXIMATELY EQUIVALENT TO EXCAVATION. IT IS CONSERVATIVE TO SAY THEY ARE EQUAL SINCE THE PROPOSED CHANNEL BED IS SIGHTLY LOW THAN EXISTING.

THEREFORE, SAY A3E = 800 CM

A3W = 1100 CM

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JOB AESP BRIDGE 10

SHEET NO. ____ OF ____

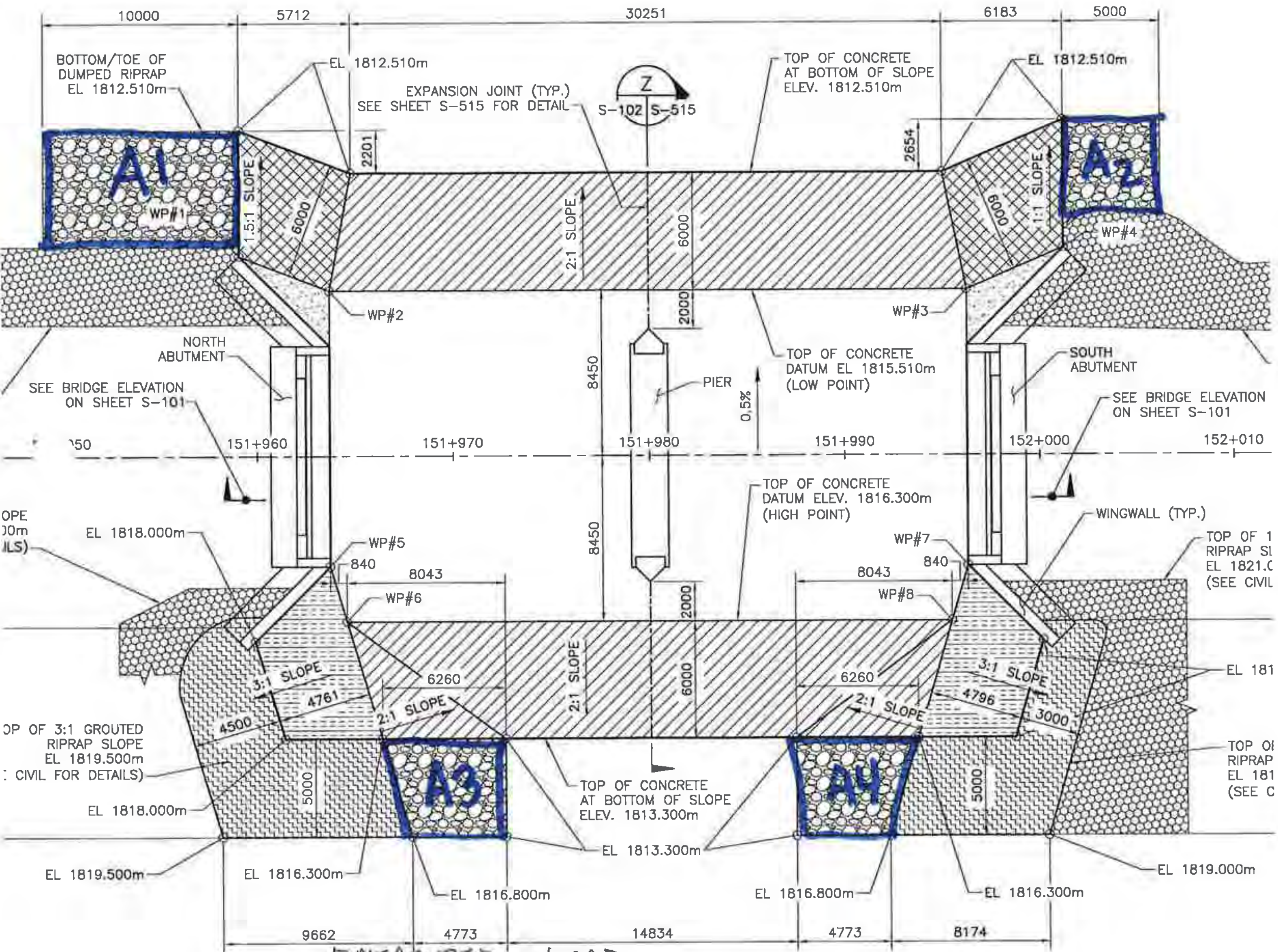
CALCULATED BY APC DATE 10/7/14

CHECKED BY ____ DATE ____

SCALE ____

RIPRAP

FOR PROTECTION OF THE SCOUR PAD.



↑ MEASURED IN CAD

	A (SM)	EL TOP (AVG)	EL BOT (AVG)	V (CM)
A1	59.5	1816.5	1814.5	119
A2	24.2	1816.0	1814.5	36
A3	27.6	1817.0	1815.0	55
A4	27.6	1817.0	1815.0	55

$\boxed{\text{TOTAL} = 270 \text{ CM}}$

Elastomeric Bearing Pads	# of Spans	Beams Per Span	Bearings Per Beam	Total Qty (#)	
Abutment and Piers					
Elastomeric Bearings	2.00	6.00	2.00	24	
Elastomeric Bearing Pad Totals				24	Ea
Say —>				**24**	Ea

Strip Seal Joint	Lane Width (m)	Number of Lanes	# Of Joints	Total Joint Length (m)	
Abutment and Piers					
Strip Seal Joint	4.05	2.00	3.00	24.3	
Asphaltic Bridge Joint Totals				24	m
Say —>				**25**	m

Bituminous Dampproofing	Length (m)	Height (m)	Area (m²)	Qty	Total Area (m²)	
Abutments						
Behind Abutment Stem	11.25	5.96	67.05	2	134.10	
Behinf Wall Stem - Southeast	7.00	6.18	43.26	1	43.26	
Behind Wall Stem - Northwest	6.00	6.41	38.45	1	38.45	
Behind Wall Stem - Northeast	6.50	6.22	40.42	1	40.42	
Behind Wall Stem - Southwest	6.00	6.28	37.68	1	37.68	
Top of Approach Slab	8.00	5.00	40.00	2	80.00	
					373.90	
Bituminous Damp-proofing Totals					374	m²
Say —>					**390**	m²

Wearing Surface & Membrane Waterproofing	Length (m)	Width (m)	Area (m²)	Qty	Total Area (m²)	
Roadway						
Asphalt Paving (50mm thick)	50.40	8.10	408.24	1	408.24	
Total					408.24	
Wearing Surface & Membrane Waterproofing Totals					408	m²
Say —>					**415**	m²

Weep Holes	# per Wall	Qty	Total Area (m²)	
Abutments				
Weep Holes	4	2	8.00	
Abutment Totals			8.00	
Walls				
Weep Holes - Southwest	1	1	1.00	
Weep Holes - Northwest	1	1	1.00	
Weep Holes - Northeast	1	1	1.00	
Weep Holes - Southeast	1	1	1.00	
Wall Totals			4.00	
Weep Hole Totals			12	m²
Say —>			**12**	m²

Scuppers	No Spouts / Each Side Span	No of Sides	No of Spans	Qty	No of Drainage Spouts
Scuppers	1.00	2.00	2.00	4	4.00
Total					4.00
Say —>					4

Cofferdam (Control/Diversion of Water)				QTY	
Cofferdam (Control/Diversion of Water)					
Cofferdam (Control/Diversion of Water)				1	
Cofferdam Totals				1	LS
Say —>				**1**	LS

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